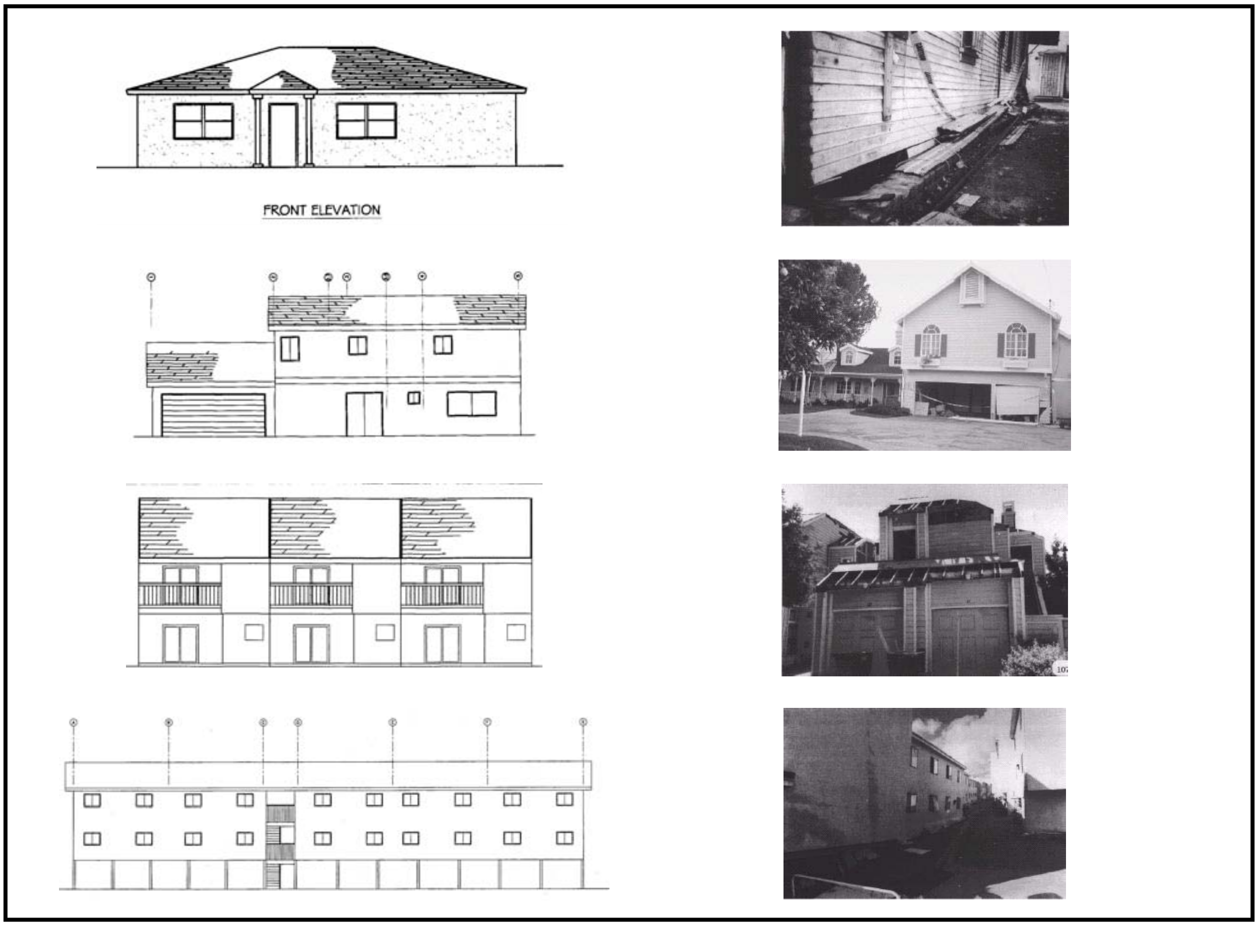
Improving Loss Estimation for Woodframe Buildings

Volume 2: Appendices



CUREE-Caltech Woodframe Project

Element 4, Economic Aspects

May 2002

Improving Loss Estimation for Woodframe Buildings

Volume 2: Appendices

Final Report on Tasks 4.1 and 4.5 of the CUREE-Caltech Woodframe Project

May 2002

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Appendix A. Literature Review

Summary

Twelve key references relevant to loss estimation for woodframe buildings are reviewed, along with the goals of the current project. The review is organized under eight topics with one to three abstracts per topic. Other relevant documents are also cited under each heading. The topics are as follows:

1. General methodologies
2. Effect of building characteristics on performance
3. Assessment of ground motion
4. Time-history structural analysis of woodframe buildings
5. Damageability of building components
6. Construction cost estimation
7. Post-earthquake habitability of buildings
8. Validation of seismic vulnerability functions

The topics are relevant to each aspect of the current project. The literature reviewed here represents state-of-the-art methods that the current project seeks either (a) to improve upon, or (b) to utilize as part of the project's development of woodframe vulnerability functions.

This project involves the development of vulnerability functions for woodframe buildings, as part of a larger program addressing seismic performance of woodframe buildings. The overall program is being overseen by the Consortium of Universities for Research in Earthquake Engineering (CUREE), and is funded by the California Governor's Office of Emergency Services (OES) and the Federal Emergency Management Agency (FEMA). For further information see www.curee.org.

General Methodologies

ATC-13

Applied Technology Council (ATC), 1985, *ATC-13: Earthquake Damage Evaluation Data for California*, Redwood City, CA: Applied Technology Council, 492 pp.

While relatively dated, this document still represents the most comprehensive compendium of vulnerability data for a wide variety of building types and other engineered facilities. The authors find that the available data on earthquake damage to some facility types are inadequate or nonexistent. Other types have adequate data to estimate damage for moderate-sized earthquakes, but not large events. The authors therefore develop seismic vulnerability functions using expert opinion gathered from 157 earthquake engineering academics and practitioners using a Delphi Process (Gordon and Helmer, 1964) modified to account for expert self-rating (Dalkey, Brown, and Cochrane, 1970).

In the ATC process, experts estimated the damage factor (repair cost as a fraction of replacement cost) to 40 categories of California buildings (as well as 38 categories of industrial, lifeline, and other assets) as a function of modified Mercalli Intensity (MMI). Damage factor probability distributions are then evaluated for each MMI level. Damage probability matrices are then created by discretizing the damage probability distributions. Seven damage-state intervals are used, which are associated with semantic damage states, from 1-None to 7-Destroyed. Several types of loss are evaluated: building repair cost, content repair cost, loss-of-use duration, minor injuries, major injuries, and fatalities. The approach is intended for macroscopic (i.e., societal) loss estimation, and does not account for a building's unique detailed design. All woodframe construction is characterized by a single vulnerability function, which is therefore useful in the present study primarily for comparison purposes.

HAZUS

National Institute of Building Sciences (NIBS) and Federal Emergency Management Agency (FEMA), 1997, *HAZUS Earthquake Loss Estimation Methodology: Technical Manual, Volumes I, II, and III, NIBS Document Number 5201,* Washington, DC: Federal Emergency Management Agency

The methodology proposed in this document uses a similar category-based approach to seismic vulnerability, and is embodied in the HAZUS software package. Like the ATC (1985) approach, HAZUS is intended for use in macroscopic loss estimation and is therefore inappropriate for single-building loss estimation. Its vulnerability functions are based on estimated fragility of three building elements: structural components, drift-sensitive nonstructural components, and acceleration-sensitive nonstructural components. Structural response under a particular earthquake is determined using the capacity spectrum method (Mahaney *et al.*, 1993), which estimates peak response by the intersection of the earthquake response spectrum and a building capacity curve that is characteristic of the model building category of interest.

Overall building damage is calculated based on the sum of estimated damage to structure, nonstructural drift-sensitive components, and nonstructural acceleration-sensitive components. Contents damage is evaluated separately. The damage state of each of these categories is treated as a random variable whose distribution is determined based on a set of four fragility curves. Each curve indicates the probability of the model building category exceeding a descriptive damage state, conditioned on spectral displacement. The fragility curves are defined in linguistic terms (slight, moderate, extensive, and complete) and are taken to be lognormal. Each damage state is equated with a damage factor (repair/replacement cost): 5% for slight, 15% for moderate, 50% for extensive, and 100% for complete. A subset of complete is collapse. The fragility-curve parameters (median and logarithmic standard deviation) are based on cyclic laboratory testing of building components and other empirical data, with results adjusted using judgment.

Assembly-Based Vulnerability

Porter, K.A., 2000, *Assembly-based Vulnerability and its Uses in Seismic Performance Evaluation and Risk-Management Decision-Making, A Doctoral Dissertation,* Stanford, CA: Stanford University, 195 pp.

The present project is required to characterize the seismic vulnerability of four particular buildings, which calls for a building-specific methodology. Porter (2000) presents the analytical framework to create building-specific seismic vulnerability functions. The methodology, called assembly-based vulnerability (ABV), is used to estimate repair cost, repair duration, and loss-of-use cost of a particular building as functions of spectral acceleration. ABV treats the building as a unique collection of standard assemblies with probabilistic fragility, repair costs, and repair durations. The procedure applies Monte Carlo methods to simulate ground motion, structural response, assembly damage, repair costs, and repair duration. The methodology is illustrated using a realistic example office building.

Various techniques are presented for developing empirical and theoretical assembly fragilities; these techniques are illustrated through the creation of fragility functions for a wide variety of structural, nonstructural, and content assemblies. The fragility functions can be reused in subsequent analyses. Fragilities are defined within the framework of a standardized, detailed, and highly adaptable assembly taxonomy (categorization system) that can facilitate unambiguous communication of assembly types, fragilities, and costs.

The study also presents a decision-analysis approach to making seismic risk-management decisions for individual buildings, using the ABV methodology. The decision analysis accounts for the decision-maker's business practices and risk attitude, and produces a recommendation of the best alternative on an expected-utility basis. A detailed procedure for eliciting the decision-maker's risk attitude is presented. The methodology is illustrated using a realistic example decision situation. It is found that risk attitude can make a material difference in the selection of the optimal risk-management alternative, thus calling into question techniques that assume risk neutrality and rely solely on cost-effectiveness as the key measure of desirability.

Other Relevant Documents

Dalkey, N., B. Brown, and S. Cochrane, 1970, "Use of Self-Ratings to Improve Group Estimates," *Technological Forecasting: An International Journal, vol. 1., no. 3.,* New York: American Elsevier Publishing, pp. 283-291

EQE International and the Geographic Information Systems Group of the Governor's Office of Emergency Services, 1995, *The Northridge Earthquake of January 17, 1994: Report on Data Collection and Analysis, Part A: Damage and Inventory Data,* Irvine, CA: EQE International, 273 pp.

EQE International and the Geographic Information Systems Group of the Governor's Office of Emergency Services, 1997, *The Northridge Earthquake of January 17, 1994: Report of Data Collection and Analysis, Part B: Analysis and Trends*, Irvine, CA: EQE International

Federal Emergency Management Agency (FEMA), 1997, *FEMA-273: NEHRP Guidelines for the Seismic Rehabilitation of Buildings,* Washington, DC: FEMA, 386 pp.

Gordon, T.J., and O. Helmer, 1964, *Report on a Long Range Forecasting Study, RAND Paper P-2982,* Santa Monica CA: RAND Corporation

Kustu, O., 1986, "Earthquake Damage Prediction for Buildings Using Component Test Data," *Proc. Third U.S. National Conference on Earthquake Engineering, Aug 24-28, 1986, Charleston, SC,* El Cerrito, CA: Earthquake Engineering Research Institute, pp. 1493-1504

Mahaney, J.A., T.F. Paret, B.E. Kehoe, et al., 1993, "The Capacity Spectrum Method for Earthquake Response During the Loma Prieta Earthquake," *Proc. 1993 U.S. National Earthquake Conference, Memphis TN, May 2-5, 1993,* Memphis, TN: Central United States Earthquake Consortium, 1993, Volume II, pp. 501-510

Panel of Earthquake Loss Estimation Methodology, 1989, *Estimating Losses from Future Earthquakes,* Washington, DC: National Research Council, 231 pp.

Porter, K.A. and A.S. Kiremidjian, 2001c, "Verifying Performance-Based Design Objectives Using Assembly-Based Vulnerability" *Structural Safety and Reliability, ICOSSAR 2001, Newport Beach, California, USA, June 17-22 2001,* Lisse, the Netherlands: A.A. Balkema.

Porter, K.A., A.S. Kiremidjian, and J.S. LeGrue, 2001b, "Assembly-based Vulnerability of Buildings and Its Use in Performance Evaluation," *Earthquake Spectra,* 17 (2), Oakland, CA: Earthquake Engineering Research Institute, 291-312.

Porter, K.A., and A.S. Kiremidjian, 2001a, *Assembly-based Vulnerability and its Uses in Seismic Performance Evaluation and Risk-Management Decision-Making,* Stanford, CA: John A. Blume Earthquake Engineering Center, 214 pp.

Reitherman, R., 1985, "A Review of Earthquake Damage Estimation Methods," *Earthquake Spectra,* 1 (4), Oakland, CA: Earthquake Engineering Research Institute, 805-847

Scholl, R.E., 1981, *Seismic Damage Assessment for Highrise Buildings, Annual Technical Report, Open-file Report 81-381,* Menlo Park, CA: US Geological Survey, 143 pp.

Steinbrugge, K.V., 1982, *Earthquakes, Volcanoes, and Tsuanmis, An Anatomy of Hazards,* New York, NY: Skandia America Group, 392 pp.

Steinbrugge, K.V., and R.J. Roth, 1995, *Dwelling and Mobile Home Monetary Losses Due to the 1989 Loma Prieta, California, Earthquake with an Emphasis on Loss Estimation,* Denver, CO: US Geological Survey, 72 pp.

Effect of Building Characteristics on Seismic Performance

Northridge Earthquake Damage Data

Comerio, M.C., 1995, Northridge Housing Losses, *A Study for the California Governor's Office of Emergency Services*, Berkeley, CA: Center for Environmental Design Research, University of California, Berkeley, 53 pp.

The study reviews data on buildings damaged by the 1994 Northridge Earthquake. The data were collected by various local jurisdictions immediately after the earthquake and compiled into a database by the California Governor's Office of Emergency Services' Geographic Information Systems (GIS) unit. The review focuses on residential buildings, with attention to number and location of post-earthquake building safety inspections, the nature of affected housing units, their post-earthquake viability for occupancy, and the demographics of their occupants. Using these data, the author examines a variety of social-policy issues that are raised by the damage data. Of particular interest for the present study is the author's examination of how the probability of earthquake damage and the severity of that damage (such as the outcome of post-earthquake safety inspections) relate to characteristics of the buildings, such as age, number of housing units per building, etc.

The author examines possible correlation of building age with the probability of earthquake damage. She finds that residential buildings were damaged in approximate proportion to their age. That is, the fraction of damaged residential buildings that were built between 1940 and 1976 is approximately equal to the fraction of all residential buildings – damaged or otherwise – that were built in the same period. Likewise, newer housing units were damaged in proportion to their prevalence in the total stock of housing units. These facts indicate that the age of a housing unit was irrelevant to the probability of its being damaged in the Northridge Earthquake.

Furthermore, the author's data suggest that there was little correlation between the severity of damage, measured in terms of the probability of the building being vacated, and the age of the building. The exception is in the category of pre-1940 construction, for which the percentage of units that were vacated significantly exceeded their prevalence in the housing stock. Finally, the author finds that the severity of earthquake damage among inspected apartments was greater than that of inspected single-family dwellings. While only 3% of inspected single-family dwellings were vacated after the earthquake, 4.6% of housing units in inspected multi-family dwellings were vacated.

Other Relevant Documents

Building and Fire Research Laboratory (BFRL), 1994, *Performance of HUD-Affiliated Properties During the January 17, 1994 Northridge Earthquake*, Gaithersburg,

MD: Building and Fire Research Laboratory, National Institute of Standards and Technology.

Earthquake Engineering Research Institute (EERI), 1990, *Earthquake Spectra, vol 6. Supplement: Loma Prieta Earthquake Reconnaissance Report,* El Cerrito CA: EERI, 448 pp.

Earthquake Engineering Research Institute (EERI), 1995, *Earthquake Spectra, Vol. 11 Supplement C: Northridge Earthquake of January 17, 1994 Reconnaissance Report: volume 2,* Oakland CA: EERI, 523 pp.

Foliente, G.C., 1994, "Modeling and Analysis of Timber Structures Under Seismic Loads," *Proceedings of a Research Needs Workshop: Analysis, Design and Testing of Timber Structures Under Seismic Loads; University of California, Forest Products Laboratory, Berkeley, September 9, 1994,* Berkeley CA: University of California, Forest Products Laboratory, pp. 87-109.

Foliente, G.C., ed., 1997, *Earthquake Performance and Safety of Timber Structures,* Madison, WI: Forest Products Society, 146 pp.

Keenan, F.J., "Earthquake Resistance of Timber Construction," *Engineering for Protection from Natural Disasters: Proceedings of the International Conference; Bangkok, January 7-9, 1980.* Karasudhi, Pisidhi, et al., eds. Chichester, England: John Wiley and Sons, pp. 273-286.

NAHB Research Center, 1994, *Assessment of Damage to Residential Buildings Caused by the Northridge Earthquake,* Washington, DC: US Department of Housing and Urban Development, 110 pp.

Scawthorn, C.R., 1982, "Optimum Seismic Design of Mid-Rise Buildings," *Proceedings of the Seventh European Conference on Earthquake Engineering, Athens, Greece, Sept. 1982,* Vol. 3, pp. 511-520

Scawthorn, C.R., H. Iemura, and Y. Yamada, 1981, "Seismic Damage Estimation for Low- and Mid-rise Buildings in Japan," *Earthquake Engineering and Structural Dynamics, vol. 9,* John Wiley & Sons, 93-115

Schierle, G.C., 2002, *Northridge Earthquake Field Investigations: Statistical Analysis of Woodframe Damage, CUREE Publication No. W-02,* Richmond, CA: Consortium of Universities for Research in Earthquake Engineering (CUREE), 117 pp.

Yancey, C.W., G.S. Cheok, F. Sadek, B. Mohraz, 1998, *Summary of the Structural Performance of Single-Family, Wood-Frame Housing, Report NISTIR-6224,* Gaithersburg, MD: National Institute of Standards and Technology, 159 pp.

Zacher, E.G., "Past Seismic Performance of Timber Buildings," *Proceedings of a Research Needs Workshop: Analysis, Design and Testing of Timber Structures Under Seismic Loads; University of California, Forest Products Laboratory, Berkeley, September 9, 1994,* Berkeley CA: University of California, Forest Products Laboratory, pp. 3-8.

Modeling of Ground Motion

SAC Ground Motions

Somerville, P., N. Smith, S. Punyamurthula, and J. Sun, 1997, *SAC/BD-97/04 Development of Ground Motion Time Histories for Phase 2 of the FEMA/SAC Steel Project,* Richmond, CA: SAC Steel Project

This report provides three sets of time histories for the Los Angeles area, each corresponding to a range of magnitudes, distances and UHS spectral amplitudes representing probabilities of 2% in 50 years, 10% in 50 years, and 50% in 50 years. These time histories represent a mixture of strike-slip, oblique, and thrust faulting and a site considered to be soil profile type D (firm soil). Some were recorded on rock and were scaled to represent a modification of the spectral shape expected for soil profile type D. Except for the simulated time histories, the time histories have not been scaled to spectrally match the target UHS for the specified probabilities, but rather an overall scaling factor was used to match over a fairly broad range of frequencies, with less weight given to the shorter periods. (Unfortunately, this is the end of the spectrum of greatest interest for the present study.) Each set of horizontal components was scaled to match the target as a mean, so each component maintains its original relationship with the others.

The results of this study are useful for present purposes to provide realistic acceleration time histories over a wide range of spectral accelerations, without undue neglect of nonlinear effects of soil properties on spectral shape. However, because of the limited number of time histories provided by this study, and the potentially larger number of simulations required for present purposes, it may be necessary to generate additional artificial time histories from these real recordings, using for example nonstationary autoregressive moving average (ARMA) methodologies as described for example by Conte et al. (1992) and Polhemus et al. (1981).

Other Relevant Documents

Conte, J.P., K.S. Pister, and S.A. Mahin, 1992, "Nonstationary ARMA Modeling of Seismic Motions," in *Soil Dynamics and Earthquake Engineering, November, 1992,* New York: Elsevier Science, pp. 411-426

Polhemus, N.W., and A.S. Cakmak, 1981, "Simulation of Earthquake Ground Motions Using Autoregressive Moving Average Models," in *Earthquake Engineering and Structural Dynamics, vol. 9,* New York: John Wiley & Sons, Inc., pp. 343-354

Structural Analysis of Woodframe Buildings

CASHEW

Folz, B., and A. Filiatrault, 2002, “Cyclic Analysis of Wood Shear Walls,” *ASCE Journal of Structural Engineering,* Reston VA: American Society of Civil Engineers

The methodology to be used in the present study requires that each index building be subjected to nonlinear dynamic structural analysis using historic or simulated ground motion time histories. When applied to woodframe buildings, this raises the challenge of modeling the load-displacement behavior of wood shearwalls based on the sheathing type and thickness, connector type and schedule, and relevant material properties. This problem is addressed by Folz and Filiatrault. The authors present a model of shearwall behavior using three structural components: rigid framing members, linear elastic sheathing panels, and nonlinear sheathing-to-framing connectors. The framing members are assumed to be pin-connected at their ends and contribute no stiffness to the overall shearwall. The properties of the sheathing panels are determined considering the shearwall geometry and material properties of the lumber. The connectors – which contribute all the shearwall’s hysteretic energy dissipation – are each modeled as two orthogonal uncoupled nonlinear springs with load-displacement behavior determined by experimental study.

The authors illustrate how the multi-degree-of-freedom system thus constituted of framing members, sheathing panels, and connectors can be analyzed to produce an equivalent nonlinear single-degree-of-freedom system, where the single degree of freedom is the horizontal displacement of the shearwall’s upper edge relative to its lower edge. The model is able to predict the experimentally obtained monotonic and cyclic load-displacement behavior of full-scale tests performed under a separate investigation. The authors have implemented the results of their research into a software package entitled CASHEW: Cyclic Analysis of SHEar Walls. CASHEW is capable of calculating the load-displacement behavior of shearwalls, but it is not intended to perform nonlinear dynamic structural analysis of entire buildings. For that aspect of the present study, one of several available software packages will be used. Two of these are documented by Prakash et al. (1992) and Carr (1996).

Other Relevant Documents

Prakash, V., G.H. Powell, and F.C. Filippou, 1992, *DRAIN-3DX: Base Program User Guide, UCB/SEMM-1992/30,* Berkeley CA: Structural Engineering, Mechanics and Materials, Dept. of Civil Engineering, Univ. of California at Berkeley, 100 pp.

Carr, A. J., 1996, *RUAUMOKO - Inelastic Dynamic Analysis Program,* University of Canterbury - Department of Civil Engineering, Christchurch, New Zealand

Damageability of Building Components

Fragility Functions

Kennedy, R.P., and M.K. Ravindra, 1984, "Seismic Fragilities for Nuclear Power Plant Risk Studies," *Nuclear Engineering and Design 79,* Philadelphia PA: Elsevier Science Publishers pp. 47-67

For many types of assemblies, component damageability can be expressed in terms of fragility, defined as the probability that some limit state is exceeded, conditioned on an input level of demand. A graph of this relationship is referred to as a fragility function. In the present study, fragility functions depict the probability that a building assembly reaches or exceeds some (damage) limit state conditioned on structural response. A characteristic fragility function is desired for each assembly type and limit state. Kennedy and Ravindra present a model for estimating the probability that a structure or equipment component will fail, conditioned on a particular level of seismic shaking intensity.

They apply their model to structure and equipment components in nuclear power plants, considering the limit state of operational failure, and the input shaking intensity measured in terms of peak ground acceleration. The authors present a 3-parameter compound lognormal fragility model, which is a lognormal distribution with median value x_m and logarithmic standard deviation β_r, but where the median x_m is itself taken as a lognormally distributed random variable with median x_{mm} and logarithmic standard deviation β_u. The three parameters of the compound lognormal are thus x_{mm}, β_r, and β_u.

This framework for component fragility can be generalized to consider other limit states and measures of shaking intensity, without affecting the underlying model of random component damageability. In the present study, the limit states are defined in terms of damage to the extent that predefined repair efforts are required. The input shaking intensity, likewise, can be characterized variously as peak transient interstory drift ratio, residual interstory drift ratio, peak floor acceleration, ductility demand, etc., as appropriate to the individual assembly under consideration. It is typically difficult to determine the two β values, so in the present study a simpler two-parameter lognormal distribution is employed, in which the median is denoted by x_m and the logarithmic standard deviation is denoted by β.

Creation of Empirical Fragility Functions

Swan, S.W., and R. Kassawara, 1998, "The Use of Earthquake Experience Data for Estimates of the Seismic Fragility of Standard Industrial Equipment," *ATC-29-1, Proc., Seminar on Seismic Design, Retrofit, and Performance of Nonstructural Components,* Redwood City CA: Applied Technology Council, pp. 313-322

Given the mathematical form of the assembly fragility function, the question is how to determine its parameters for a particular assembly and limit state. Three approaches to determining the parameters are available: empirical, theoretical, and judgment-based. The empirical method involves estimating the parameters through regression analysis of earthquake experience or laboratory data. Swan and Kassawara describe such a methodology using an empirical dataset, gathered primarily from earthquake experience. In their examples, the limit state considered is operational failure. Capacity is assumed to be lognormally distributed and measured in terms of peak ground acceleration (PGA). In their approach, failure data are binned by PGA. That is, one compiles a list of components observed to have experienced known levels of PGA, and groups these components in bins of similar PGA. The number of components in each bin that failed is divided by the total number of components in the bin, producing an estimate of the fraction of components that fail when subjected to that level of PGA. One plots this fraction versus bin-average PGA. Through an appropriate transformation, linear regression analysis is performed to determine x_m and β.

In cases where available empirical data are inadequate to perform a regression analysis, under certain circumstances a theoretical component fragility function can be created. Such an approach is described by Porter (2000, described above).

Other Relevant Documents

Behr, R.A., A. Belarbi, and C.J. Culp, 1995, "Dynamic Racking Tests of Curtain Wall Glass Elements with In-Plane and Out-of-Plane Motions," in *Earthquake Engineering and Structural Dynamics, vol. 24,* New York: J. Wiley & Sons, Inc., pp. 1-14

Behr, R.A., and C.L. Worrell, 1998, "Limit States for Architectural Glass Under Simulated Seismic Loadings," in *Proc., Seminar on Seismic Design, Retrofit, and Performance of Nonstructural Components, ATC-29-1 January 22-23, 1998, San Francisco,* Redwood City, CA: Applied Technology Council, pp. 229-240

Kanvinde, A.M., and G.G. Deierlein, 2000, "Review and Generalization of Hysteresis Models for Gypsum Wall Analysis," *Proc., CUREE-Caltech Woodframe Project, Element 1 – Research Meeting, September 15-16, 2000, University of California, San Diego,* Richmond CA: Consortium of Universities for Research in Earthquake Engineering (CUREE)

Oliva, M.G., 1990, *Racking Behavior of Wood-framed Gypsum Panels Under Dynamic Load, Report No. UBC/EERC-85/06,* Berkeley CA: Earthquake Engineering Research Center, 49 pp.

Pantelides, C.P., and R.A. Behr, 1994, "Dynamic In-Plane Racking Tests of Curtain Wall Glass Elements," in *Earthquake Engineering and Structural Dynamics, vol. 23,* New York: J. Wiley & Sons, Inc., pp. 211-228

Pardoen, G.C., 1999, "Testing and Analysis of Woodframe Structures in the United States," *CUREE Publication No. W-01, Proceedings of the Invitational Workshop* *on Seismic Testing, Analysis and Design of Woodframe Construction,* Richmond CA: Consortium of Universities for Research in Earthquake Engineering (CUREE), pp. 73-79

Construction Cost Estimation

Construction Cost Estimation Using Assemblies

RS Means Co., 2000a, *Means Assemblies Cost Data,* Kingston, MA: RS Means Co.

The methodology to be employed in the present study requires estimation of the cost to repair damaged assemblies. Standard cost-estimation procedures will be used. One of the most familiar and commonly used cost-estimation systems in the United States is documented by the RS Means Co. In summary, one itemizes a construction project as a set of quantities of predefined individual assemblies to be constructed, applies a unit cost per assembly, and sums the result to find an overall direct cost of construction. A location factor is then applied to account for local variation in labor and material costs, and costs for fees, overhead, and profit are added to estimate the total construction cost.

In this system, assemblies are defined and categorized based on an extension of the UniFormat system (American Society for Testing and Materials, 1996), which describes each building assembly as belonging to one of twelve major divisions, numbered from 1 to 12, generally in the same order as the order of construction in a new building. The RS Means Co. extends this division numbering with additional levels of classification: a numerical subdivision, a major classification, and an individual line number. As a result, assemblies are defined at a highly detailed level. For example, two distinct types of stucco wall are 4.5-110-2100 (Stucco wall, cement stucco, 7/8-in. thick plywood sheathing, stud wall, 2” x 4”, 16” O.C.), and 4.5-110-2200, which is the same as 4.5-110-2100 except that the studs are 24” O.C.

Each assembly is assigned a numerical code, a detailed description of the assembly is provided, and an average construction cost is determined via an annual survey of construction contractors. The authors provide construction costs per unit, which they also define appropriately for the assembly type under consideration. For example, the two types of stucco wall mentioned above are provided with construction costs per square foot.

This system can be applied to the repair of existing buildings by considering that repair tasks are the same as new construction with some additional work required, notably protection from heat and dust, demolition and disposal of damaged components, and line-of-site repair of adjacent, undamaged components. Line-of-site issues arise because undamaged components may require repainting or other upgrade to match the conditions of newly repaired assemblies. These line-of-site costs can be estimated using another major cost-estimation methodology documented in RS Means Co. (2000b). This approach is further documented by Wetherill (1989).

Other Relevant Documents

American Society for Testing and Materials (ASTM), 1996, "E1557-96 Standard Classification for Building Elements and Related Sitework – UNIFORMAT II," *1997 Annual Book of ASTM Standards, Section 4, Construction, Volume 04.11 Building Constructions,* West Conshohocken, PA: ASTM, pp. 630-639

RS Means Co., 2000b, *Means Construction Cost Data,* Kingston, MA: RS Means Co.

Wetherill, E.B., 1989, *Means Repair and Remodeling Estimating,* Kingston, MA: R.S. Means Corp., 452 pp.

Post-Earthquake Habitability of Buildings

ATC-20

Applied Technology Council (ATC), 1991, *ATC-20: Procedures for Postearthquake Safety Evaluation of Buildings,* Redwood City, CA: ATC, 144 pp.

The present study requires commentary on relating damage to post-earthquake habitability of the dwellings. Two documents with widespread familiarity among the engineering community address this issue. The most well-known of these is ATC-20, which presents procedures and guidelines that are used by most cities and other jurisdictions after an earthquake has occurred to assess building safety. The procedures, developed for use by structural engineers and building department officials, provide for both rapid and detailed safety evaluations.

Under the rapid-evaluation procedure, any one of six conditions makes a building unsafe to occupy. The first of these is that the building has "collapsed, partially collapsed, or moved off its foundation." Second, the "building or any story is significantly out of plumb." In practice, this means a residual drift in excess of 1 to 2 in. per story, or as little as 0.7% residual drift ratio. Third, there is "obvious severe damage to primary structural members, severe racking of walls, or other signs of severe distress." In practice for a woodframe structure, this condition can be difficult to determine and apply, as relatively brittle stucco and other finish materials can be severely damaged by transient drifts without involving significant damage to the woodframe structural system.

The fourth condition indicates an unsafe area in or near a building: "obvious parapet, chimney, or other falling hazard." In the present application, this condition could include severe damage to a brick chimney or to an attached roof structure such as a porch canopy. The fifth condition has to do with ground failure, as opposed to evidence of building damage: "Large fissures in ground, massive ground movement, or slope displacement present." The last condition is likewise unconnected to building damage: "Other hazard present (e.g., toxic spill, asbestos contamination, broken gas line, fallen power line)." A section of the report provides more-detailed guidelines specific to woodframe structures.

FEMA-273

Federal Emergency Management Agency (FEMA), 1997, *FEMA-273: NEHRP Guidelines for the Seismic Rehabilitation of Buildings,* Washington, DC: FEMA, 386 pp.

In contrast with ATC-20's post-earthquake safety criteria, this document prescribes building-component performance criteria for use in pre-earthquake safety evaluations. FEMA-273 addresses many of the same safety issues as ATC-20, but because of its nature as an analytical tool, it also deals with features that would not be readily apparent during a post-earthquake inspection.

To be considered by FEMA-273 to be safe for immediate occupancy, a building must "substantially retain its original strength and stiffness," a characteristic that can be determined by a structural analysis, but might be difficult to observe in practice. Minor cracking of facades, partitions, ceilings, and structural elements is allowed. Distributed minor hairline cracking of gypsum and plaster veneers is allowed. Wood diaphragms may not have observable loosening or withdrawal of fasteners, or splitting of sheathing or framing. Minor damage to canopies, chimneys, stairs, light fixtures, and doors is allowed.

Some of the performance criteria in FEMA-273 are stricter than those of ATC-20. For example, one criterion for immediate occupancy is that the building can have no broken window panes. It seems unlikely that in a post-earthquake situation a broken window in an otherwise undamaged dwelling would render the home uninhabitable in the eyes of the tenant or a reasonable building-department official. This criterion could probably be relaxed for the present application. Another example is that, for a building to be occupiable immediately after an earthquake, it must have permanent drift less than 0.25%, a stricter criterion than that of ATC-20.

Validation of Seismic Vulnerability Functions

ATC-38

Applied Technology Council (ATC), in press, *ATC-38: Database on the Performance of Structures Near Strong-Motion Recordings,* Redwood City, CA: Applied Technology Council

ATC-38 comprises a survey of 530 buildings located within 300 meters of strong-motion recording sites that were strongly shaken in the Northridge Earthquake. Ground shaking at these sites, measured in terms of peak ground acceleration, ranged from 0.20g to 1.8g. The surveys were performed by teams of two licensed civil or structural engineers, with each survey taking approximately two person-hours per building. Of 183 woodframe buildings observed, more than half appear to have been inspected inside and out, as opposed to exterior-only ("sidewalk") surveys. Survey data included structure type, some structural and nonstructural design and configuration characteristics, geotechnical effects, and a variety of damage and casualty measures that would not be available from other sources such as building permits. Detailed data with photographs are provided in a relational database (several formats), and some preliminary data reduction and correlation studies are included. Repair costs were not recorded, and information on structural damage hidden beneath architectural finishes was unavailable to the surveyors.

The ATC-38 study is reviewed because it will probably be difficult to validate the seismic vulnerability functions resulting from the current study. Validation implies a test that the results of the study adequately predict some observable state of nature. Overall results of the study will therefore be difficult to test, as the index buildings are hypothetical, and will not be subjected to real ground shaking. Furthermore, comparison with other theoretical vulnerability functions, such as those described in other literature reviewed above, do not constitute a predictive test. It may be possible to use damage data from the 1994 Northridge Earthquake to check the ability of the present study to postdict damage on an order-of-magnitude basis. Comparison with theoretical results will therefore have to be made based on predicted versus observed damage state of some important building feature such as residual drift or functionality.

It may also be possible to compare the relative damageability of poor-quality versus typical and superior construction using detailed building characteristics documented in the ATC-38 database, e.g., using the "deterioration of structure" field that appears in one of the tables to distinguish poorly maintained buildings.

Appendix B. Index Buildings

(See http://www.curee.org for Autocad drawings)

Figure B-1:
Small House Floor Plan (Sheet A-1, Right)

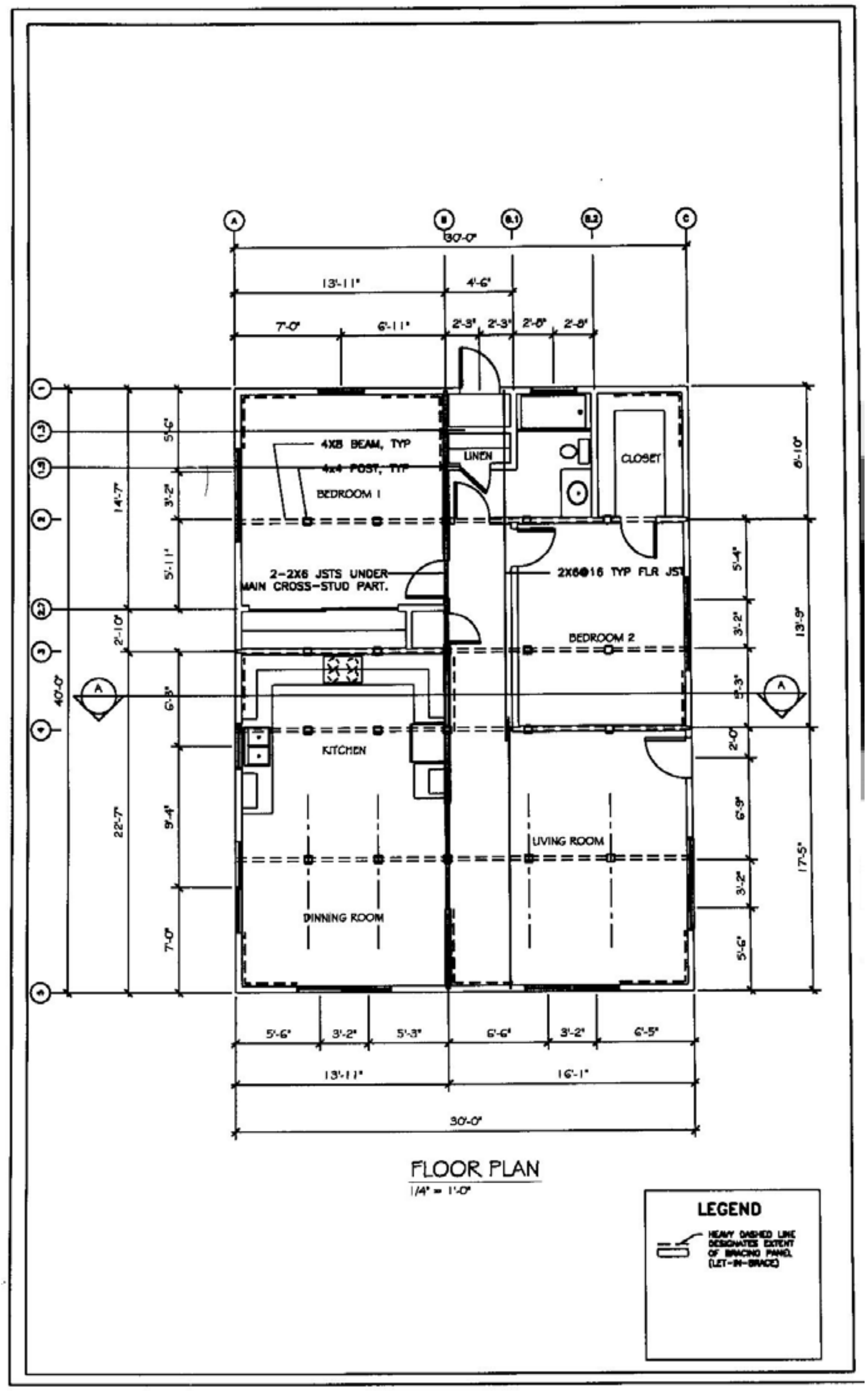


Figure B-2:
Small House Roof Plan And Section A-A (Sheet A-1, Left)

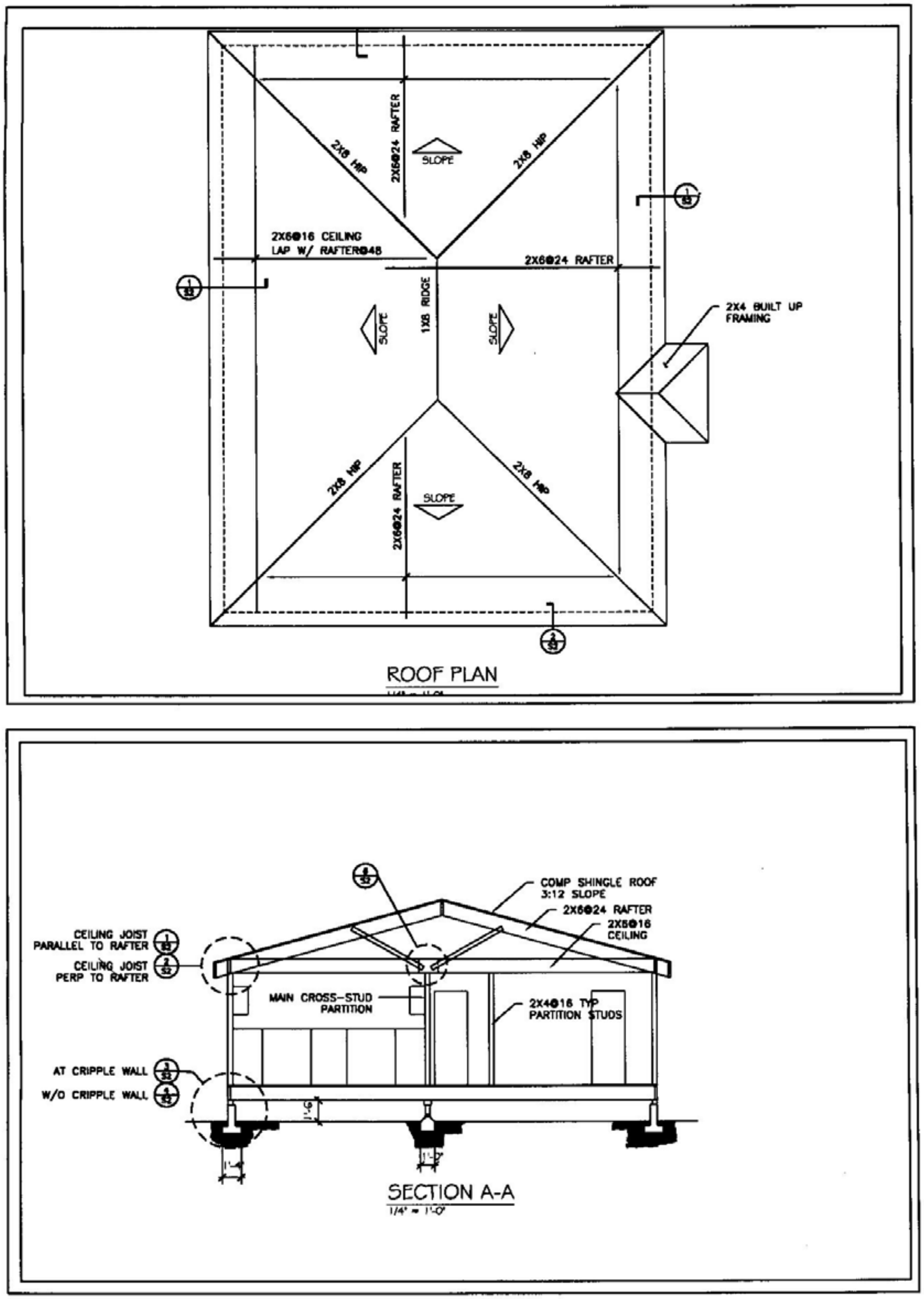
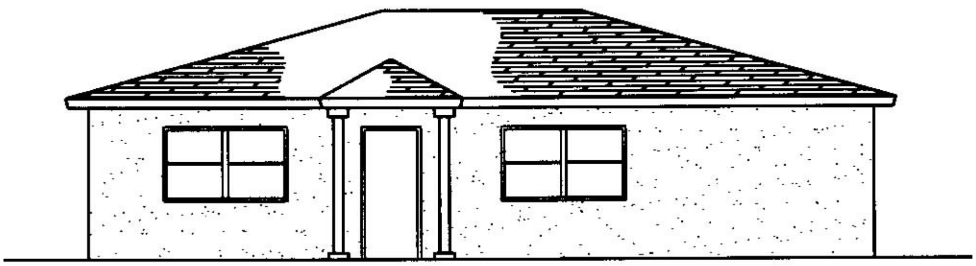
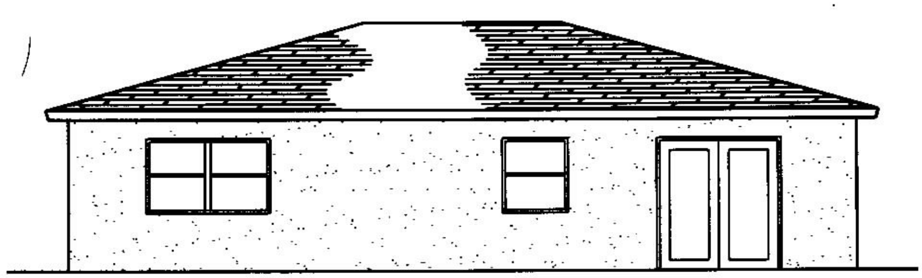


Figure B-3:
Small House Exterior Elevations (Sheet A-2, Right)

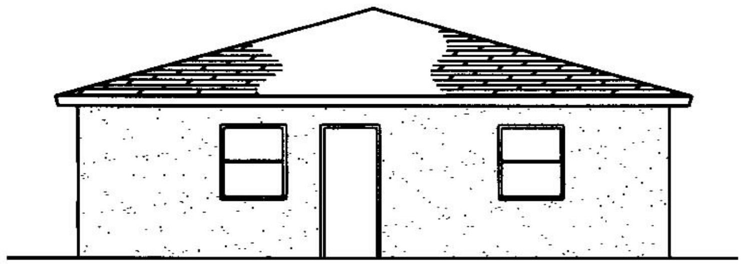


FRONT ELEVATION

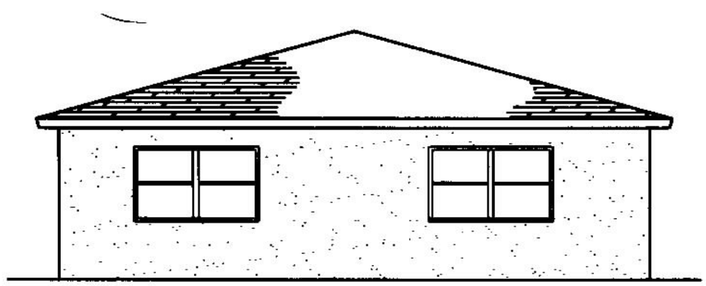


REAR ELEVATION

Figure B-4:
Small House Exterior Elevations (Sheet A-2, Left)



RIGHT ELEVATION



LEFT ELEVATION

Figure B-5:
Small House Structural Notes (Sheet S-1, Right)

NOTES FOR SMALL HOUSE INDEX BUILDING

The Small House Index Building is one of four index buildings prepared for Element 4 – Economic Aspects, as part of the CUREe–Caltech Woodframe Project. This house is being used as part of structural analysis studies conducted under Task 1.5.4, which will be used for component-based fragility analyses conducted under Task 4.1.

Drawing sheets A1, A2, S1 and S2 contain information describing the small house. Sheets A1 and A2 contain the architectural plans, drafted by Roy Young and Associates, based on a description developed by the Woodframe Project. Descriptions of framing have been added to Sheet A1 by Element 3. Sheets S1 and S2 contain typical details, notes, assembly and weight information, provided by Element 3.

The small house is a one story single family dwelling of 1200 square feet. It is intended to have been built as a housing development "production house" in the early 1950's, located in either Northern or Southern California. The design is based on prescriptive construction. To the extend possible, characteristic materials and fastening have been identified.

SPECIES
Typical species for framing – Douglas-fir.
Foundation sill plates – heart redwood.

SHEATHING
Roof sheathing 1x6 straight sheathing boards
2-8d common nails per crossing
8d common dimensions from Bethlehem Steel match ASTM F1667:
Flat head, diamond point, L=2.5", D=.131"

Floor sheathing – same as roof sheathing, applied diagonally. Note that the diagonal application leads to edge nailing and approximately 3" on center for all edges for the floor diaphragm.

Gypsum wallboard sheathing – 3/8" sheathing – probably applied vertically (by the late 50's horizontal placement was being encouraged).
1958 UBC describes two nails:
1. Smooth shank--Flat-head diamond point 5d – 13-1/2 gage, 1-5/8 long
2. Deformed shank –slightly countersunk head –.098" x 1-1/4" long x 1/4 dia. Head.
Nail 1 is slightly longer and larger diameter than described for F1667 NLGWS-05. The second nail matches fairly closely F1667 NLGWM-02.
Bethlehem steel describes a nail to match number 1. It was provided cement coated and was a non-stock size.
For gypsum board wall sheathing one of these fasteners would have been used at 6-8 inches on center (say 7) over the height of each stud. We understanding is that they would not have been edge-nailed at this spacing to the top or bottom plates. The spacing at top and bottom plates would likely have been 16 inches on center as part of the vertical line of fasteners at each stud.

Gypsum board ceiling sheathing. The fasteners above would have been spaced at 5 to 7 inches along the ceiling joists. The perimeter edges parallel to the joists would have been nailed in order to provide proper vertical support. The edges perpendicular would not have been nailed.

Note that there was some discussion as the whether interior wall and ceiling finish would have been button board with a plaster finish coat. It was decided that while this construction was still common in custom homes in this period, production homes would more typically have been built with gypsum wall board.

STUCCO
UBC Sec. 4710. 18 ga woven wire, furred out from backing 1/4" nailed with galvanized nails, ?" minimum penetration, spaced 6" maximum vertically, 16" horizontally.

FASTENING

Anchor bolts – 1/2 inch diameter at 6'-0" maximum on center.

Framing nailing is thought to have been mostly done with common nails, although box nails were allowed by code. The following is the schedule of minimum fastening from the 1958 UBC

Joist to sill or girder – toe nail 2-16d
Bridging to joist – toe nail 2-8d
1x6 subfloor to joist – face nail 2-8d
2-inch subfloor to joist or girder 2-16d
Plate to joist or blocking 16d – 16"oc
Stud to plate – end nail 2-16d
Stud to plate – toe nail 3-16d or 4-8d
Top plates –spike together 16d – 24"oc
–laps and intersections 2-16d
Ceiling joists –to plate – toe nail 2-16d
–laps over partitions 3-16d
–to parallel alternate rafters 3-16d
Rafter to plate 3-16d
Continuous 1-inch brace to stud 2-8d
2-inch cut-in brace to stud 2-16d
1-inch sheathing to bearing 2-8d
Corner studs and angles 16d – 30"oc*
(Built-up corners)

*Reference c is more specific in showing corner configurations and calls for 16d-16"oc between studs or 3-10d into each spacer block.

Additional Fastening From Reference c, Figure 7

Rim joist (called header) to joists – end nails 2-20dRim joist to sill – toe nails 10d – 12"oc

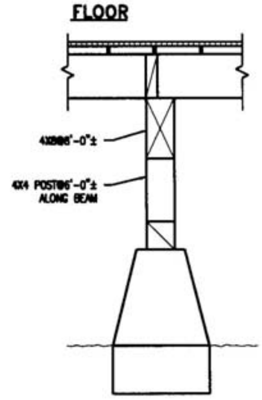
LET-IN-BRACES
The floor plan of the small house shows likely locations of 2x4 let-in braces. Because the finish materials are thought to have a much larger effect on the building behavior, the let in braces may of may not be included in computer modeling of the house.

REFERENCES

a. Uniform Building Code, 1958 Edition
b. Bethlehem Wire Nails and Other Wire Products, Bethlehem Steel, 1950.
c. Technique of House Nailing, Housing and Home Finance Industry, Washington D. C., November 1947.
d. ASTM F-1667-95 Standard Specification for Driven Fasteners: Nails, Spikes, Staples

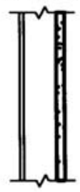
Figure B-6:
Small House Structural Notes (Sheet S-1, Left)

FLOOR



	PSF
FLOORING	1.0
1X SHEATHING (ACTUAL 3/4")	2.2
2X6@16" (1-5/8X5-1/2)	1.6
BLOCKING	0.3
4X8@6'-0" AVG (3-5/8X5-1/2)	0.8
TOTAL	5.9

EXTERIOR WALL



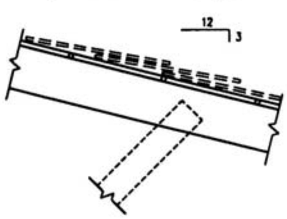
	PSF
7/8" 3 COAT STUCCO	8.8
2X4 STUDS@16" (1-5/8X3-1/2)	1.1
TOP PLATES	0.3
INT FINISH MIN 1/2" GYP	1.8
TOTAL	12.0

1/4" GLAZING + FRAME ~ 4.0 PSF

INTERIOR WALL

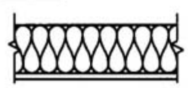
	PSF
2X4 STUDS@16"	1.1
TOP PLATES	0.3
FINISH - 2 X 1/2" GYPBD	3.6
TOTAL	5.0

3:12 SLOPED ROOF



	PSF
COMP SHINGLE	3.5
RE-ROOF	3.5
BUILDING PAPER	
1X SHEATHING (ACTUAL 3/4)	2.2
2X6@24" (1-5/8X5-1/2)	1.1
2X4@24" TO 1/4 POINTS	0.7
MISC	0.0
TOTAL	11.0
ADJUST FOR SLOPE X1.0308	11.3

CEILING



	PSF
2X6@16" (1-5/8X5-1/2)	1.6
INSULATION 0.3 #/inX3-1/2	1.1
FINISH MIN 5/8" GYP	2.3
MISC	0.0
TOTAL	5.0

Small House Index Building

Building Weights

Item	Location		Length (ft)	Width or Height (ft)	Area (sq ft)	Unit Weight (psf)	Weight #	Total Weight #
Roof			32	42	1344	11	14,784	14,784
Ceiling			30	40	1200	5	6,000	6,000
Wall	Line A-Rear	Total	40	8	320			3,344
		window/door			62	4	248	
		typ exterior			258	12	3,096	
	Line B-Int	Total	40	8	320			1,360
		open & door			80	2	160	
		typ interior			240	5	1,200	
	Line B.1-Int	Total	19	8	152			710
		open & door			16.7	2	33	
		typ interior			135.3	5	677	
	Line B.2-Int	Total	8	8	64			320
		open & door			0	2	0	
		typ interior			64	5	320	
	Line C-Front	Total	40	8	320			3,256
		window/door			73	4	292	
		typ exterior			247	12	2,964	
	Line 1-Right	Total	30	8	240			2,544
		window/door			42	4	168	
		typ ext			198	12	2,376	
	Line 1.3-Int	Total	5	8	40			200
		open & door			0	2	0	
		typ interior			40	5	200	
	Line 1.6-Int	Total	5	8	40			150
		open & door			16.7	2	33	
		typ interior			23.3	5	117	
	Line 2-Int	Total	18	8	144			620
		open & door			33.4	2	67	
		typ interior			110.6	5	553	
	Line 2-Int	Total	18	8	144			467
		closet door			63.3	1	63	
		typ interior			80.7	5	404	
	Line 3-Int	Total	14	8	112			560
		open & door			0	2	0	
		typ interior			112	5	560	
	Line 4-Int	Total	12	8	96			480
		open & door			0	2	0	
		typ interior			96	5	480	
	Line 5-Left	Total	30	8	240			2,544
		window/door			42	4	168	
		typ ext			198	12	2,376	
Floor			30	40	1200	5.9	7080	7080
Cripple Walls	Line A	ext	40	2	80	9.9	792	792
	Line C	ext	40	2	80	9.9	792	792
	Line 1	ext	30	2	60	9.9	594	594
	Line 5	ext	30	2	60	9.9	594	594
Total Building Weight								47,190

Figure B-7:
Small House Structural Details (Sheet S-2)

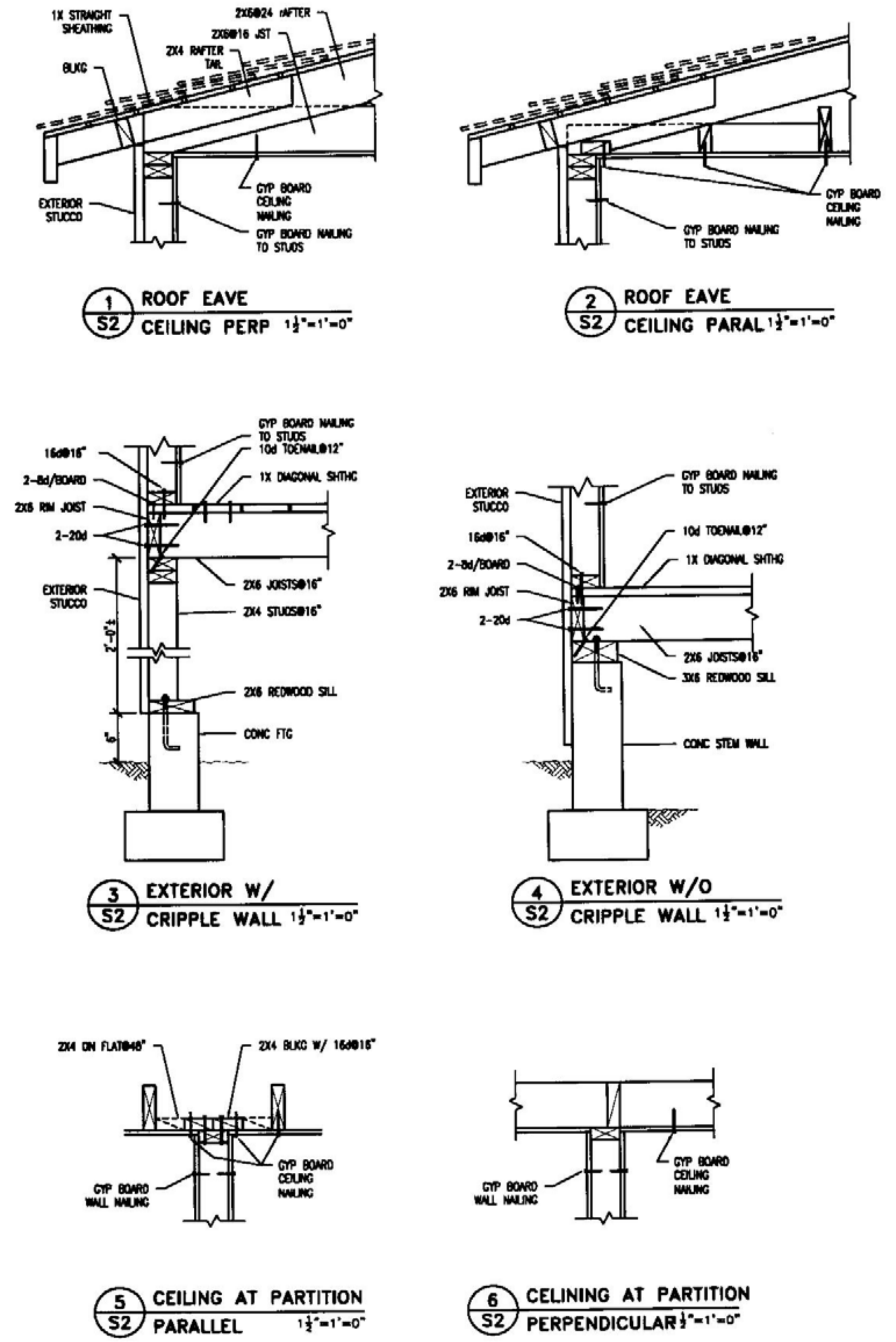


Figure B-8:
Large House Plans (Sheet A-1, Left)

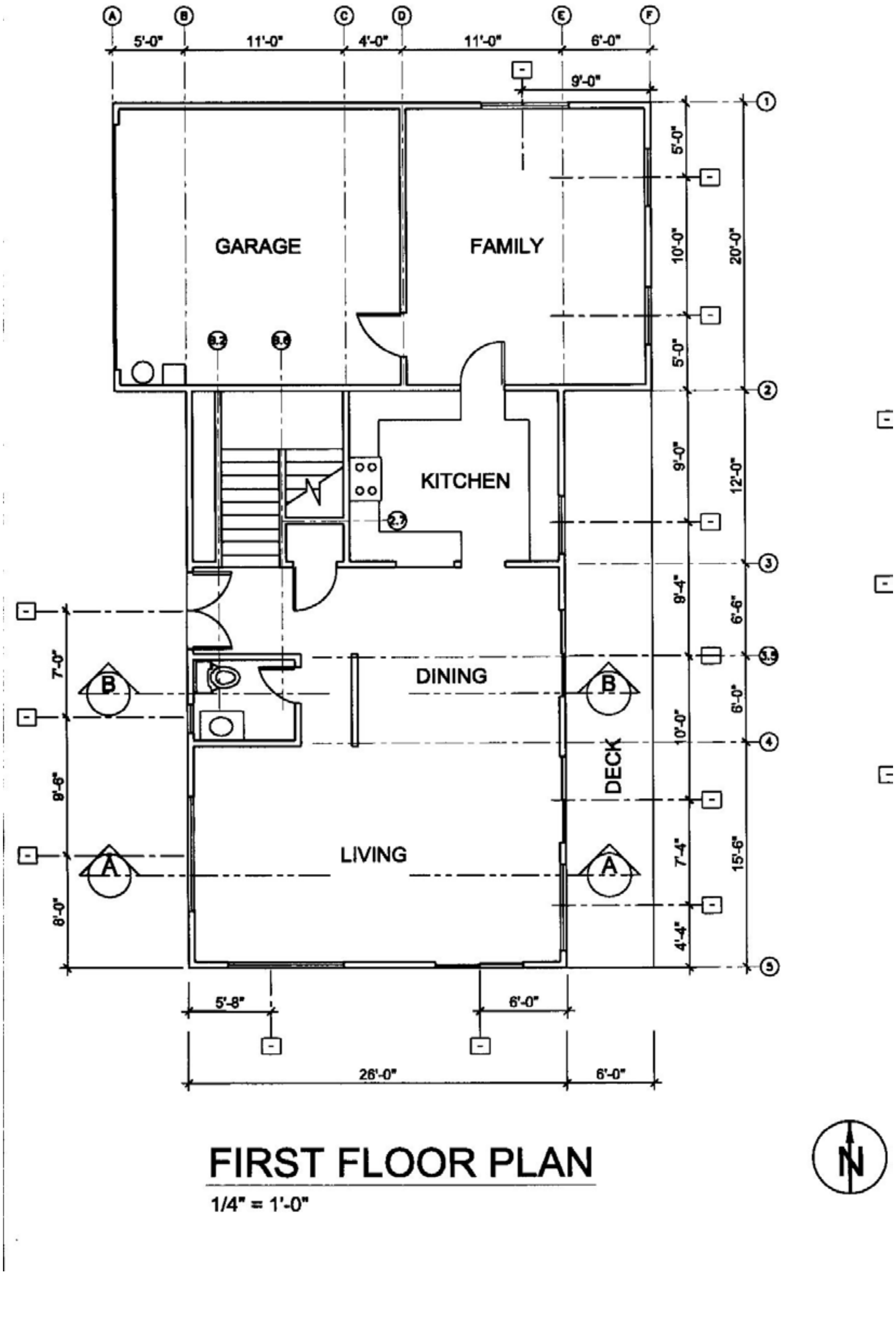


Figure B-9:
Large House Plans (Sheet A-1, Right)

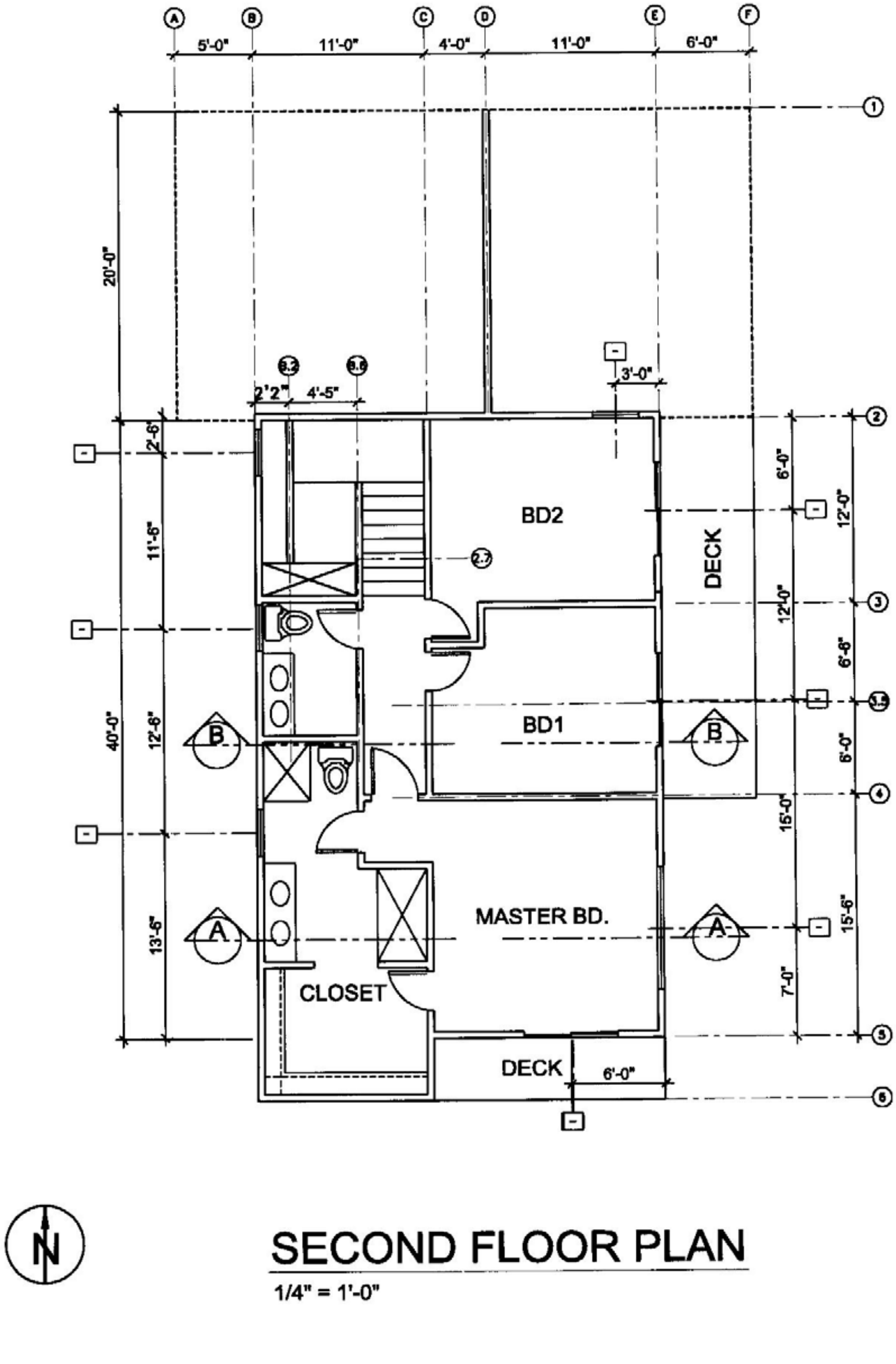


Figure B-10:
Large House Exterior Elevations (Sheet A-2, Right)

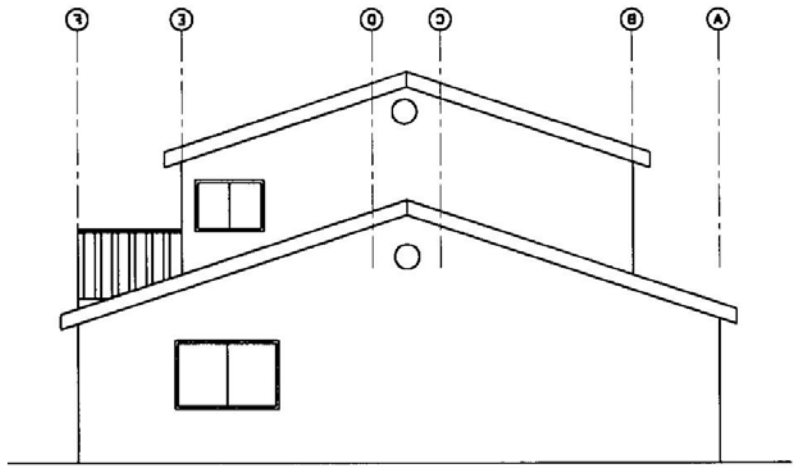
WEST ELEVATION

1/4" = 1'-0"

EAST ELEVATION

1/4" = 1'-0"

Figure B-11:
Large House Exterior Elevations (Sheet A-2, Left)



NORTH ELEVATION
1/4" = 1'-0"

SOUTH ELEVATION
1/4" = 1'-0"

Figure B-12:
Large House Roof Plan And Sections (Sheet A-3, Left)

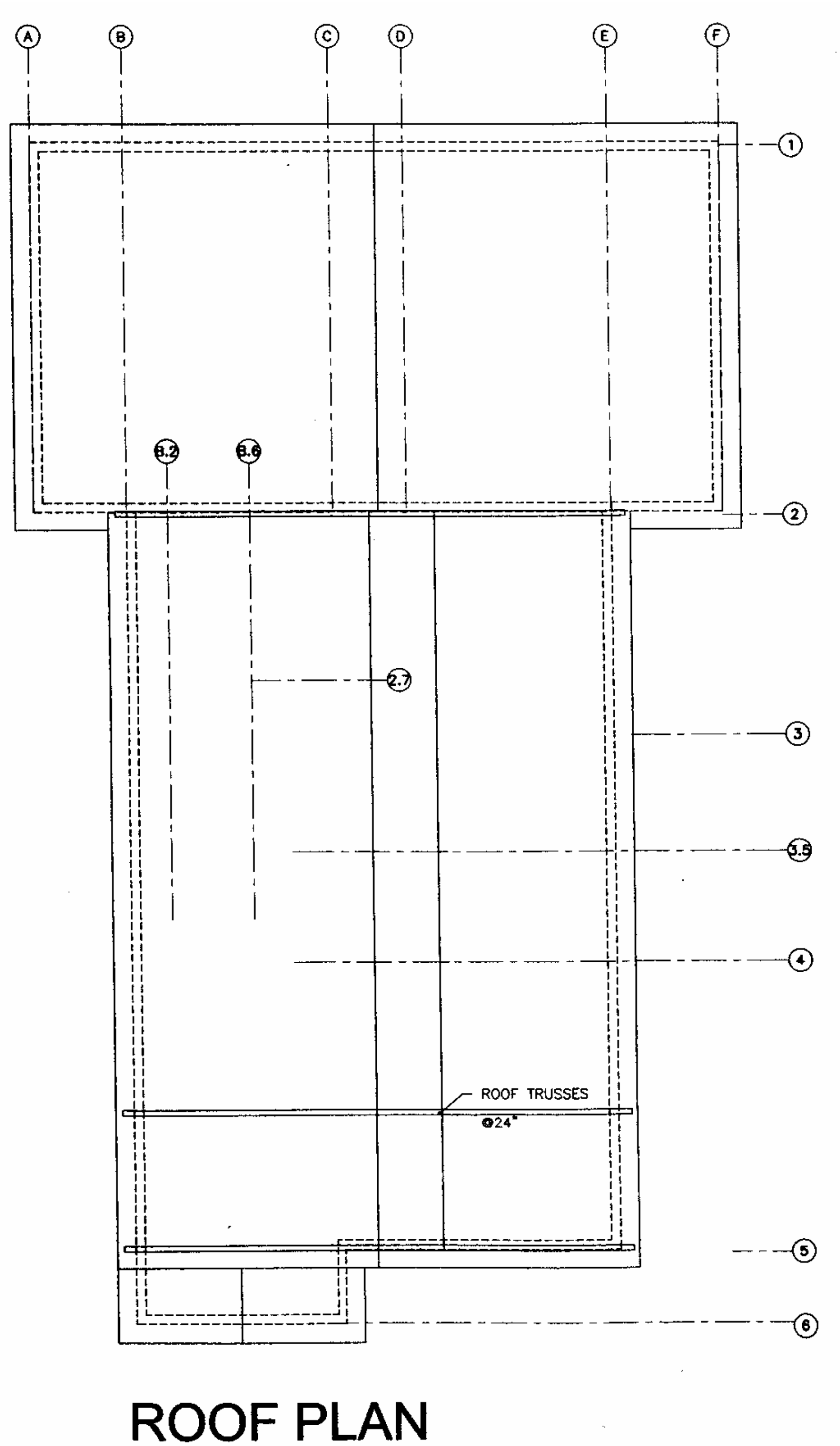


Figure B-13:
Large House Roof Plan And Sections (Sheet A-3, Right)

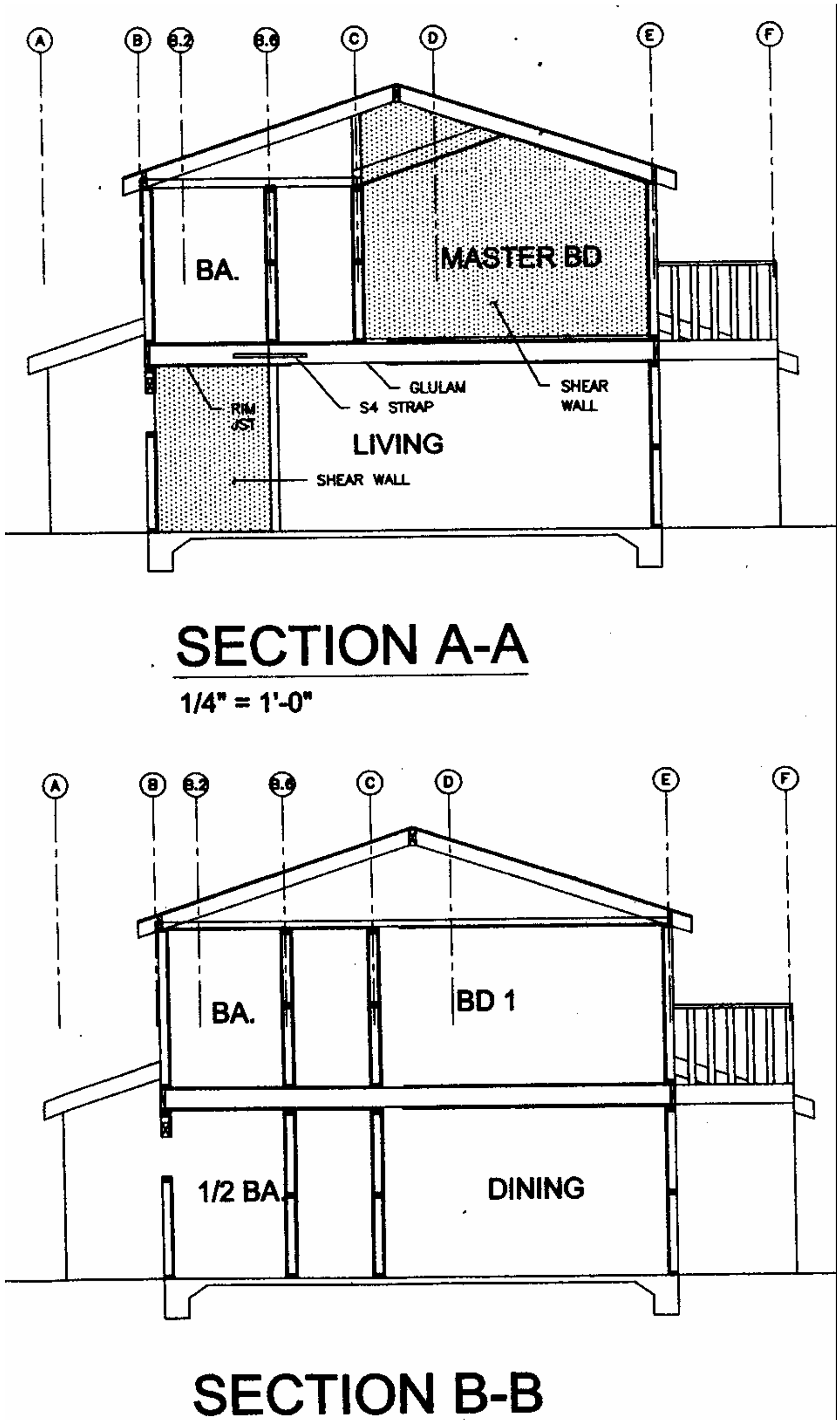
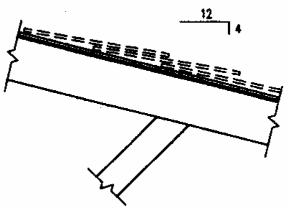


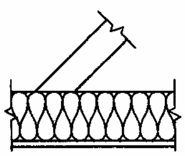
Figure B-14:
Large House Structural Notes (Sheet S-1, Left)

4:12 SLOPED ROOF



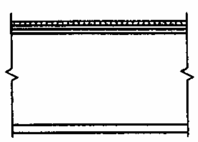
	PSF
CONC TILE	10.0
RE-ROOF	0.0
BUILDING PAPER	
1/2" PLWD (7/16 OSB)	1.5
2X6@24" TRUSS TOP CHORD	1.0
2X4@24" TRUSS WEB	0.6
SUB-TOTAL	13.1
ADJUST FOR SLOPE X1.054	13.8
MISC	0.2
TOTAL	14.0

CEILING



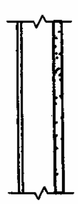
	PSF
2X4@24" BOTTOM CHORD	0.6
2X4@24" TRUSS WEB	0.6
INSULATION 0.3 #/inX3-1/2	1.1
FINISH MIN 5/8" GYP	2.3
MISC	0.2
TOTAL	4.8

FLOOR



	PSF
FLOORING	1.0
3/4"T&G PLWD (OR OSB)	2.2
2X12@16"	3.1
5/8" GYPBOARD	2.3
TOTAL	8.6

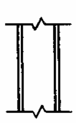
EXTERIOR WALL



	PSF
7/8" 3 COAT STUCCO	8.8
2X4 STUDS@16"	1.1
TOP PLATES	0.3
INT FINISH MIN 1/2" GYP	1.8
TOTAL	12.0

1/4" GLAZING + FRAME ~ 4.0 PSF

INTERIOR WALL



	PSF
2X4 STUDS@16"	1.1
TOP PLATES	0.3
FINISH - 2 X 1/2" GYPBD	3.6
TOTAL	5.0

WOOD SHEAR WALL SCHEDULE

SYMBOL	ALLOWABLE SHEAR (PLF)	SHEATHING MATERIAL	MIN STUD AT ADJOINING PANEL EDGES	FOUNDATION SILL	SHEATHING EDGE NAILS (1)	SHEATHING INTERMEDIATE NAILS (1)	ANCHOR BOLTS (2)
6	260	3/8 C-D PLWD OR 7/16 OSB SHEATHING	2x	2x	8d@6"	8d@12"	5/8"@32"
4	380		2x	2x	8d@4"		5/8"@32"
3	490		2x	2x	8d@3"		5/8"@16"
2	NA		3x	2x	8d@2"		5/8"@16"

1. Common nails, substitutions must be approved by Engineer.
2. Minimum two bolts per piece of sill.

TIEDOWN SCHEDULE

SYMBOL	TIEDOWN	ALLOWABLE UPLIFT (#)	ADJUSTED* ALLOWABLE UPLIFT (#)
P18	PA18	2155	NA
P28	PA28	3685	NA
H2	HD2	3265	NA
H5	HD5	4385	NA
S1	ST22	1215	550
S2	ST6224	1875	940
S3	ST6236	2475	1650
S4	MST48	3345	2510

* ADJUSTMENT ONLY CONSIDERS NAILS FALLING IN POST ABOVE & BELOW. NAILS AT FLOOR DEPTH ARE DEDUCTED.

Figure B-15: Large House Structural Notes (Sheet S-1, Right)

The Large House Index Building is one of four index buildings prepared for Element 4 – Economic Aspects, as part of the CUREe–Caltech Woodframe Project. This house is being used as part of structural analysis studies conducted under Task 1.5.4, which will be used for fragility analyses conducted under Task 4.1.

Drawing sheets A1–A3 and S1–S3 contain information describing the small house. Sheets A1–A3 contain the architectural plans, drawn by Ray Young and Associates, based on a description developed by the Woodframe Project. Sheets S1–S3 contain typical details, notes, assembly and weight information, provided by Element 3.

The large house is a two story single family dwelling of approximately 2400 square feet. It is intended to have been built as a housing development "production house" in either the 1980's or 1990's, located in either Northern or Southern California. The design is based on engineered construction. To the extend possible, characteristic materials and fastening have been identified.

SPECIES
Typical species for framing – Douglas–fir.
Foundation sill plates – pressure treated Hem–Fir.
Roof trusses – could vary – assume Southern Pine.

SHEATHING
Roof sheathing 15/32 OSB
8d common at 6" supported edges, 12" other supports.
8d common dimensions – ASTM F1667:
Flat head, diamond point, L=2.5", D=.131"

Floor sheathing 3/4" T&G PLWD or OSB
10d common at 6" / 12".

Wall sheathing 7/16" OSB.
8d common. See shear wall schedule for edge nail
spacing. 12" field spacing.

Gypsum wallboard sheathing – ½" sheathing – probably applied horizontally.
1988 UBC describes cooler nails. Per ASTM F1667 these are:
Smooth shank---Flat–head diamond point 5d – 13–1/2 gage, 1–5/8 long

For gypsum board wall sheathing one of these fasteners are at 7 inches on center over the height of each stud. We understanding is that they would not have been edge–nailed at this spacing to the top or bottom plates. The spacing at top and bottom plates would likely have been 16 inches on center as part of the vertical line of fasteners at each stud.

Gypsum board ceiling sheathing. The fasteners above are at 7 inches along the ceiling joists. The perimeter edges parallel to the joists would have been nailed in order to provide proper vertical support. The edges perpendicular would not have been nailed.

STUCCO
UBC Tables 47B & C. 18 ga hexagonal woven wire, furred out from backing ¼" nailed with 16 gauge staple, 7/8" minimum leg, spaced 6" maximum vertically, 16" horizontally.

FASTENING

Anchor bolts – per shear wall schedule at noted shear walls, otherwise ½ inch diameter at 6'–0" maximum on center.

Framing nailing was generally done with coated sinker nails. The following is the schedule of minimum fastening from the 1988 UBC. It needs to be kept in mind that much of the framing nailing was done with 16d sinker gun nails. Fastening noted as having multiple 8d nails were more likely to have one or two 16d sinker nails.

1. Joist to sill or girder – toe nail 3–8d
2. Bridging to joist – toe nail each end 2–8d
6. Sole plate to joist or blocking – face nail 16d @ 16"
7. Top plate to stud – end nail 2–16d
8. Stud to sole plate – toe nail 2–16d end or 4–8d toe
9. Double studs – face nail 16d @ 24"
10. Doubled top plates – face nail 16d @ 16"
11. Top plates –laps and intersections face nail 2–16d
13. Ceiling joists – to plate – toe nail 3–8d
14. Continuous header to stud – toe nail 4–8d
15. Ceiling joists – laps over partitions face nail 3–16d
16. Ceiling joists to parallel rafters face nail 3–16d
17. Rafter to plate – toe nail 3–16d
21. Corner studs and angles (built up corners) 16d @ 24"
22. Built–up corners and beams 20d @ 32" at top and bottom staggered, 2–20d at ends and each splice.

REFERENCES
a. Uniform Building Code, 1988 Edition
b. ASTM F–1667–95 Standard Specification for Driven Fasteners: Nails, Spikes, Staples

Figure B-16:
Large House Structural Details (Sheet S-2, Left)

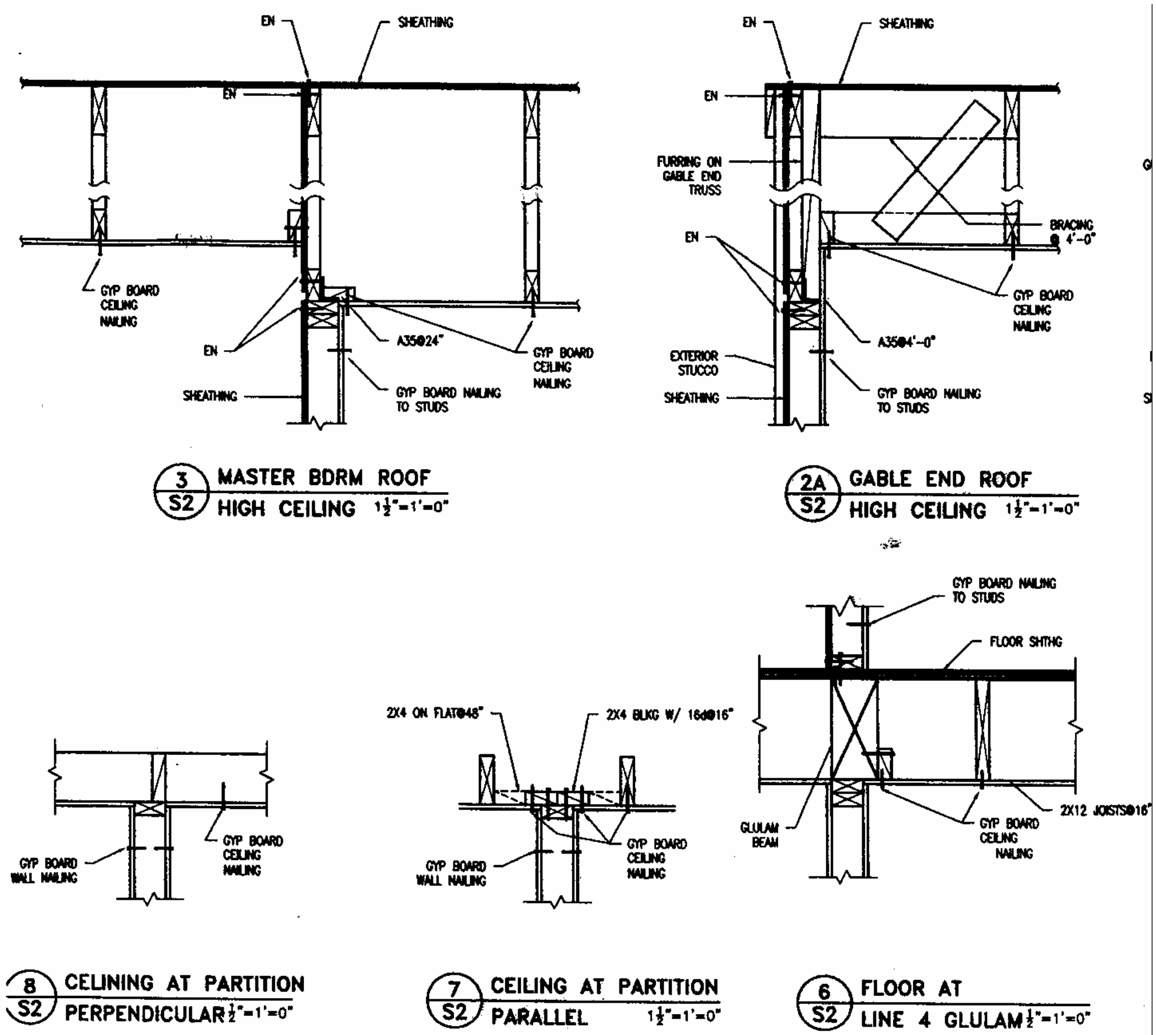


Figure B-17:
Large House Structural Details (Sheet S-2, Right)

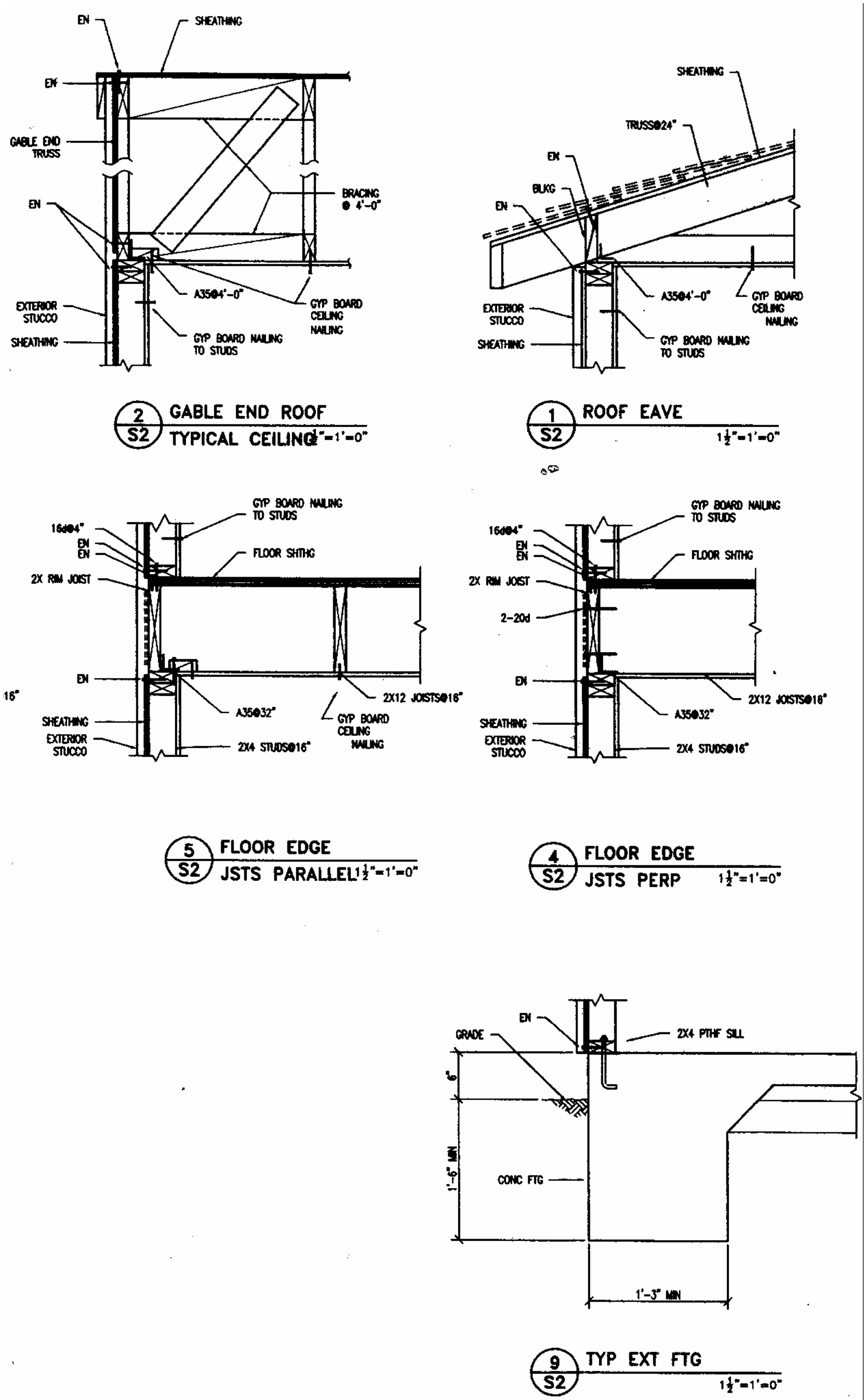


Figure B-18:
Large House Structural Plans (Sheet S-3, Left)

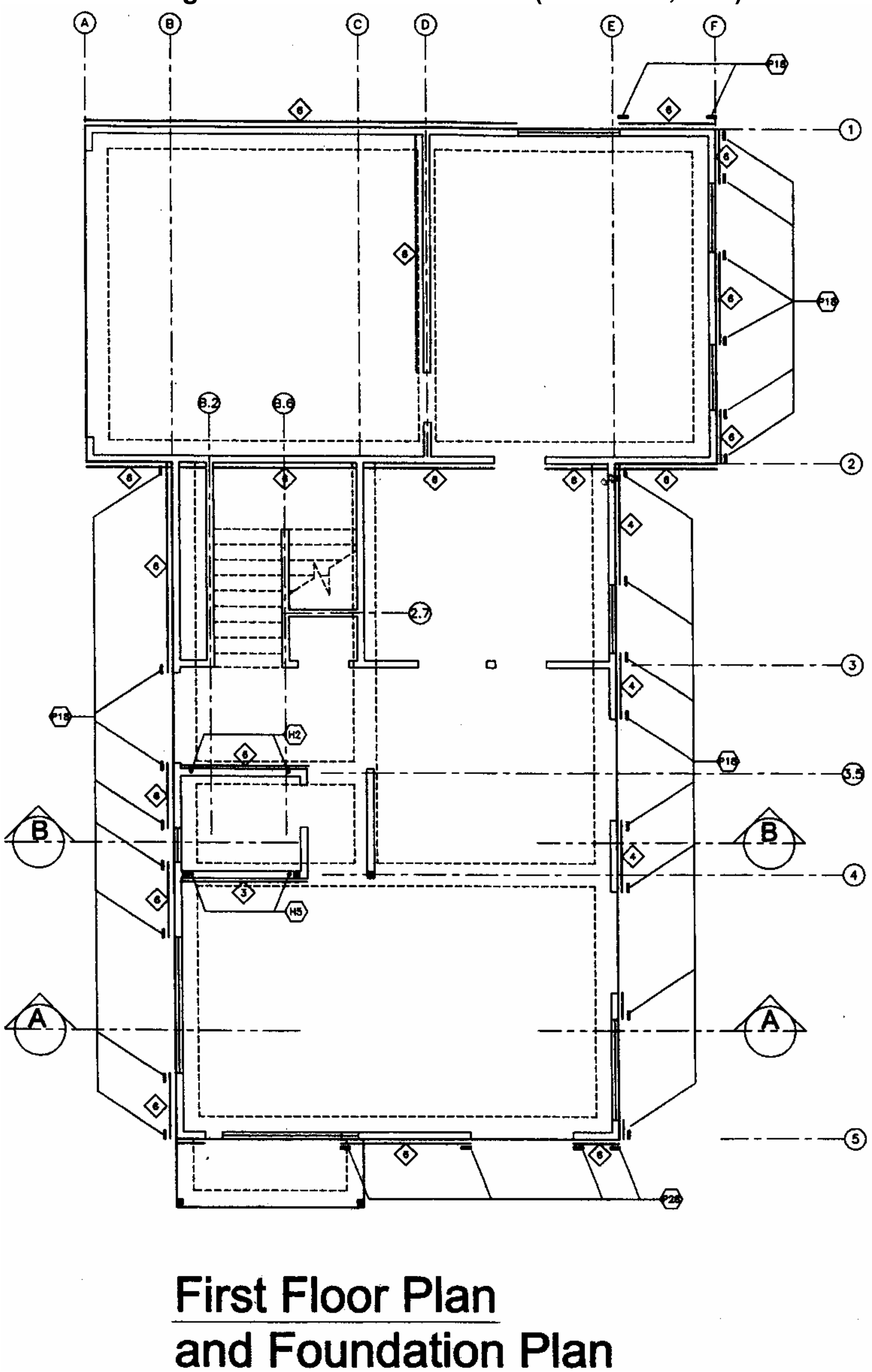


Figure B-19:
Large House Structural Plans (Sheet S-3, Right)

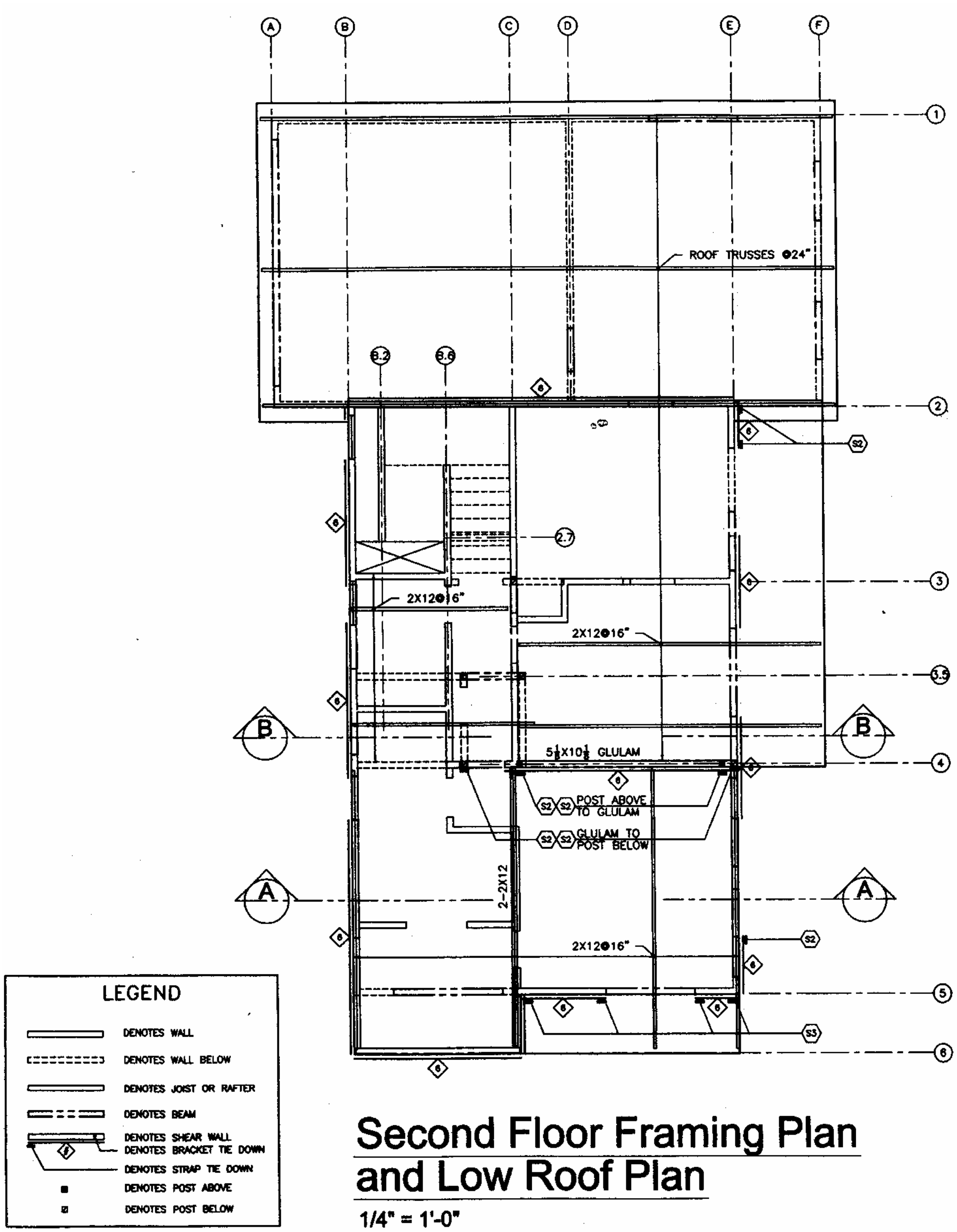


Figure B-20:
Townhouse Location of Unit 3 (Sheet A-3 Right)

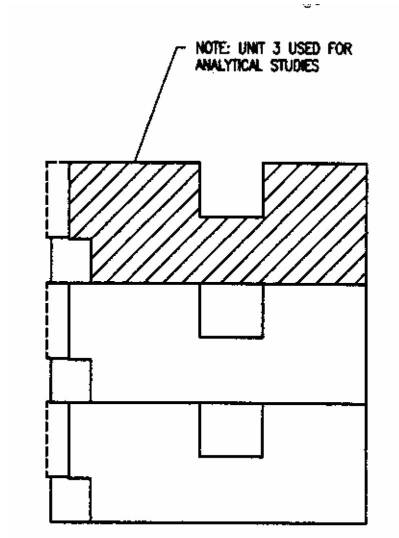


Figure B-21:
Townhouse Unit 3 Plans (Sheet A-3 Right)

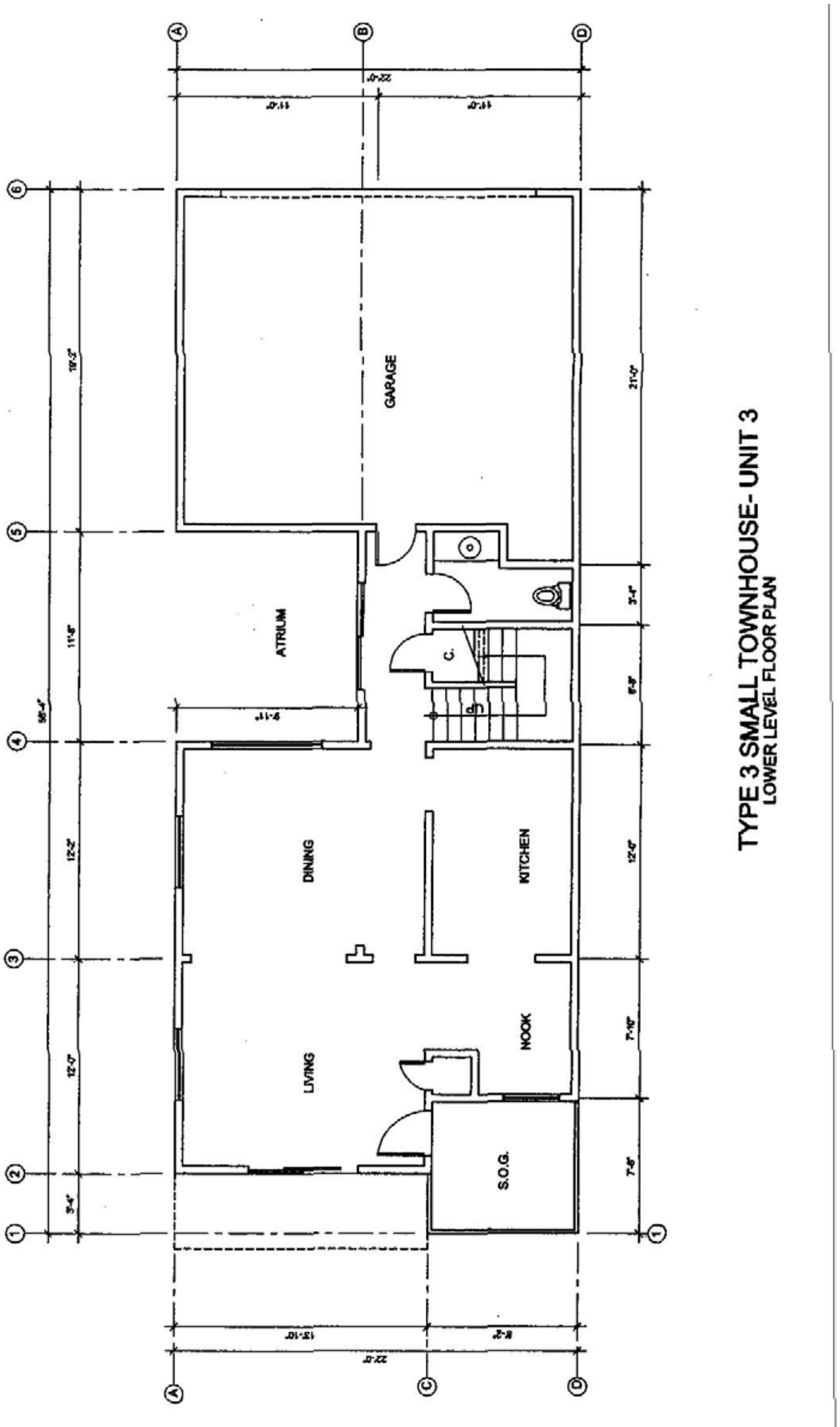


Figure B-22:
Townhouse Unit 3 Plans (Sheet A-3 Left)

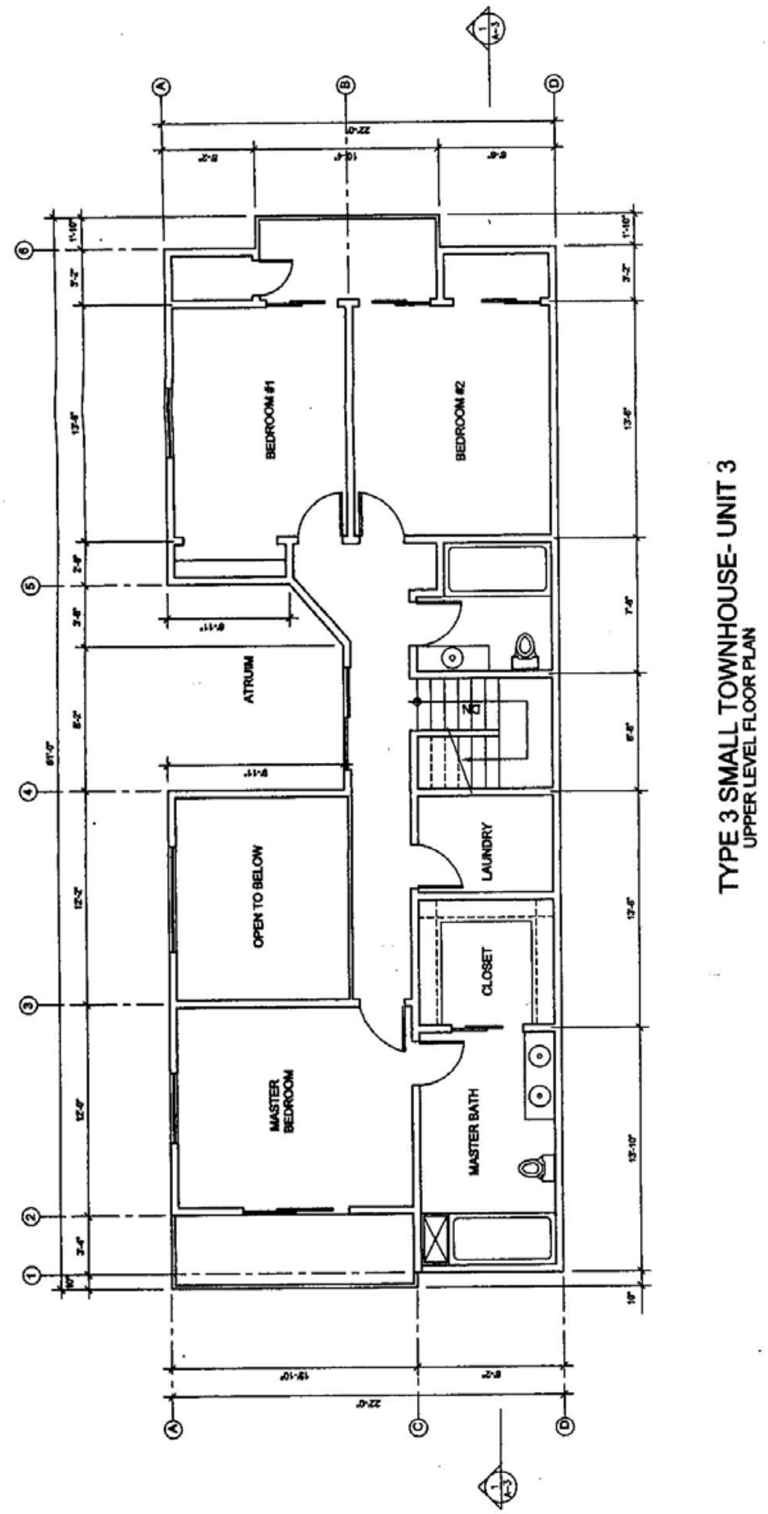


Figure B-23:
Townhouse North (L) And South (R) Elevations (Sheet A-4)

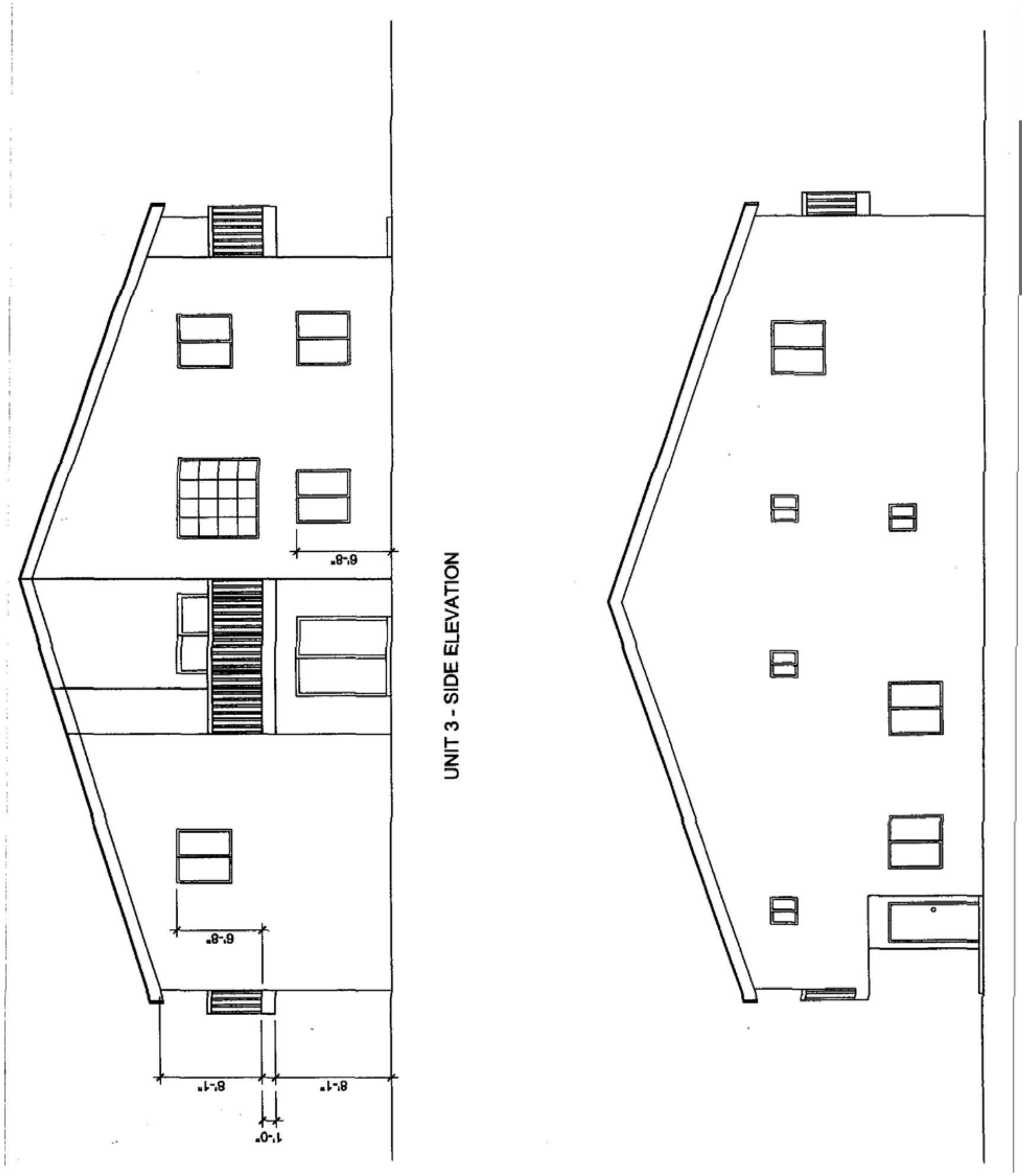
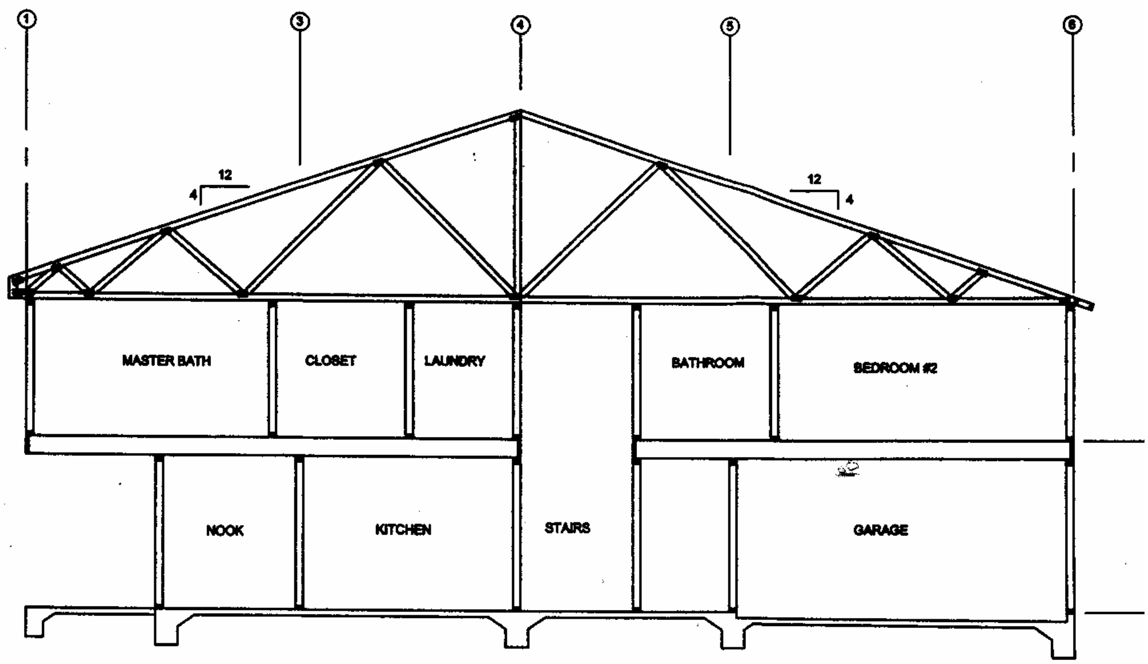
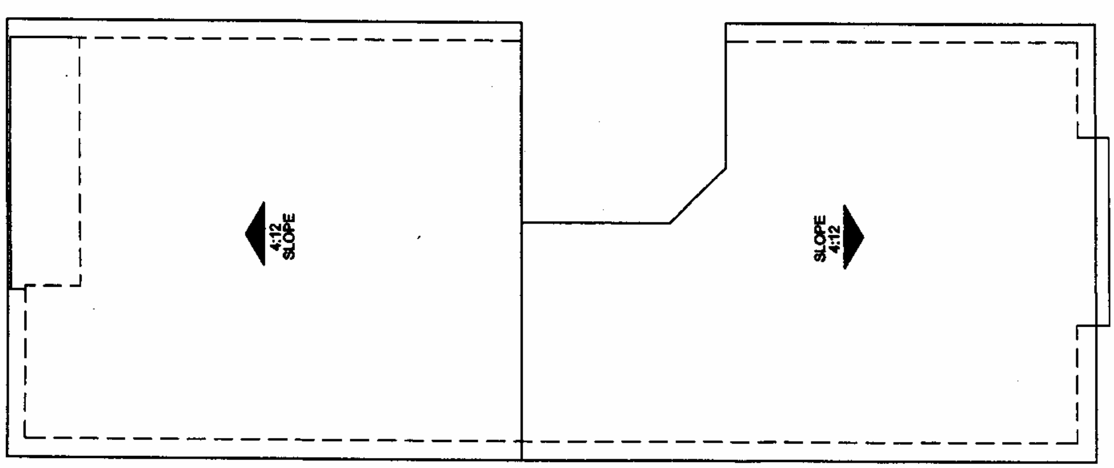


Figure B-24:
Townhouse West (L) And East (R) Elevations (Sheet A-5)

Figure B-25: Townhouse Roof Plan And Section (Sheet A-6)



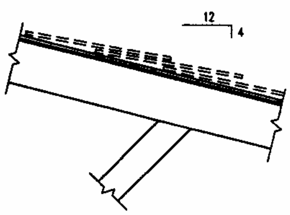
TYPE 3 SMALL TOWNHOUSE SECTION - 1



ROOF PLAN

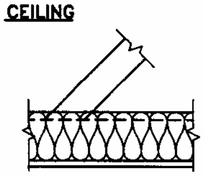
Figure B-26:
Townhouse Structural Notes (Sheet S-1, Left)

4:12 SLOPED ROOF



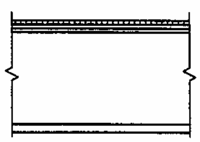
	PSF
CONC TILE	10.0
RE-ROOF	0.0
BUILDING PAPER	
1/2" PLWD (7/16 OSB)	1.5
2X6@24" TRUSS TOP CHORD	1.0
2X4@24" TRUSS WEB	0.6
SUB-TOTAL	13.1
ADJUST FOR SLOPE X1.054	13.8
MISC	0.2
TOTAL	14.0

CEILING



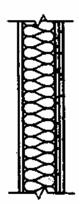
	PSF
2X4@24" BOTTOM CHORD	0.6
2X4@24" TRUSS WEB	0.6
INSULATION 0.05 #/inX6	0.3
FINISH MIN 1/2" GYP	1.8
MISC	0.2
TOTAL	3.5
ADD @ STUCCO SOFFIT	6.0

FLOOR



	PSF
FLOORING	1.0
3/4"T&G PLWD (OR OSB)	2.2
2X12@16"	3.1
GYPBOARD	2.2
MISC.	0.5
TOTAL	9.0
ADD @ STUCCO SOFFIT	6.0

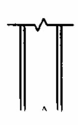
EXTERIOR WALL



	PSF
7/8" 3 COAT STUCCO	8.8
2X4 STUDS@16"	1.1
INSULATION	0.2
PLATES	0.5
3/8" PLYWOOD-OSB	1.1
INT FINISH MIN 1/2" GYP	1.8
MISC.	0.2
TOTAL	13.6

1/4" GLAZING + FRAME ~ 4.0 PSF
DOORS ~ 2.0 PSF

INTERIOR WALL



	PSF
2X4 STUDS@16"	1.0
PLATES	0.5
FINISH - 2 X 1/2" GYPBD	3.6
MISC.	0.2

SHEAR WALL SCHEDULE

SYMBOL	ALLOWABLE SHEAR (PLF)	SHEATHING MATERIAL	MIN STUD AT ADJOINING PANEL EDGES	FOUNDATION SILL	SHEATHING EDGE NAILS (1)	SHEATHING INTERMEDIATE NAILS (1)	ANCHOR BOLTS (2)
6	260	3/8 C-D PLWD OR 7/16 OSB SHEATHING	2x	2x	8d@6"	8d@12"	1/2"@48"
4	380		2x	2x	8d@4"		1/2"@32"
3	490		2x	2x	8d@3"		1/2"@16"
2			3x	2x	8d@2"		1/2"@16"
S1	180	STUCCO	2x	2x	5d@6"		1/2"@4"
G1	100	GYP. BD.	2x	2x	5d@7"		1/2"@6"

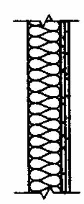
1. Common nails, substitutions must be approved by Engineer.
2. Minimum two bolts per piece of sill.

TIEDOWN SCHEDULE

SYMBOL	TIEDOWN	ALLOWABLE UPLIFT (#)	ADJUSTED* ALLOWABLE UPLIFT (#)
P18	PA18	2155	NA
P28	PA28	3685	NA
H2	HD2	3265	NA
H5	HD5	4385	NA
S1	ST22	1215	560
S2	ST6224	1875	940
S3	ST6236	2475	1850
S4	MST48	3345	2510

* ADJUSTMENT ONLY CONSIDERS NAILS FALLING IN POST ABOVE & BELOW. NAILS AT FLOOR DEPTH ARE DEDUCTED.

PARTY WALL



	PSF
2X4 STUDS@16"	1.0
INSULATION	0.2
PLATES	0.5
3/8" PLYWOOD	1.1
1/2" DRYWALL FINISH	1.8
MISC.	0.2
TOTAL	4.8
ADD AT GARAGE FOR 5/8" DRYWALL	0.4

Figure B-27:
Townhouse Structural Notes (Sheet S-1, Right)

NOTES FOR LARGE HOUSE INDEX BUILDING

The Townhouse Index Building is one of four index buildings prepared for Element 4 – Economic Aspects, as part of the CUREe-Caltech Woodframe Project. This house is being used as part of structural analysis studies conducted under Task 1.5.4, which will be used for fragility analyses conducted under Task 4.1.

Drawing sheets A1-A3 and S1-S3 contain information describing the townhouse. Sheets A1-A3 contain the architectural plans, drawn by Ray Young and Associates, based on a description developed by the Woodframe Project. Sheets S1-S3 contain typical details, notes, assembly and weight information, provided by Element 3.

The Townhouse is a two story single family dwelling of approximately 1630 square feet with an attatched two car garage. It is intended to have been built as a housing development "production house" in either the 1980's or 1990's, located in either Northern or Southern California. The design is based on engineered construction. To the extend possible, characteristic materials and fastening have been identified.

SPECIES
Typical species for framing – Douglas-fir.
Foundation sill plates – pressure treated Hem-Fir.
Roof trusses – could vary – assume Douglas-fir.

SHEATHING
Roof sheathing 15/32 OSB
8d box at 6" supported edges, 12" other supports.
8d common dimensions – ASTM F1667:
Flat head, diamond point, L=2.5", D=.131" pneumatically installed.

Floor sheathing 3/4" T&G PLWD or OSB
10d skrew shank pneumatically installed at 6" / 10".

Wall sheathing 7/16" OSB.
8d common. See shear wall schedule for edge nail
spacing. 12" field spacing.

Gypsum wallboard sheathing – ½" sheathing – applied horizontally.
Smooth shank--parker-head diamond point 5d – 13-1/2 gage, 1-5/8 long

For gypsum board wall sheathing these fasteners are at 7 inches on center over the height of each stud. We understanding is that they would not have been edge-nailed at this spacing to the top or bottom plates. The spacing at top and bottom plates would likely have been 16 inches on center as part of the vertical line of fasteners at each stud.

Gypsum board ceiling sheathing. The fasteners above are at 7 inches along the ceiling joists. The perimeter edges parallel to the joists would have been nailed in order to provide proper vertical support. The edges perpendicular would not have been nailed or blocked.

STUCCO
UBC Tables 47B & C. 18 ga hexagonal woven wire, furred out from backing ¼" nailed with 16 gauge staple, 7/8" minimum leg, spaced 6" maximum vertically, 16" horizontally,except at perimeter of wall 6"o.c. spacing.

FASTENING

Anchor bolts – per shear wall schedule at noted shear walls, otherwise ½ inch diameter at 6'-0" maximum on center.

Framing nailing was generally done with coated sinker nails. The following is the schedule of minimum fastening from the 1988 UBC. It needs to be kept in mind that much of the framing nailing was done with 16d sinker nails. Fastening noted as having multiple 8d nails were more likely to have one or two 16d sinker nails.

1. Joist to sill or girder – toe nail 3-8d
2. Bridging to joist – toe nail each end 2-8d
6. Sole plate to joist or blocking – face nail 16d @ 16"
7. Top plate to stud – end nail 2-16d
8. Stud to sole plate – toe nail 2-16d end or 4-8d toe
9. Double studs – face nail 16d @ 24"
10.Doubled top plates – face nail 16d @ 16"
11.Top plates –laps and intersections face nail 2-16d
13.Ceiling joists – to plate – toe nail 3-8d
14.Continuous header to stud – toe nail 4-8d
15.Ceiling joists – laps over partitions face nail 3-16d
16.Ceiling joists to parallel rafters face nail 3-16d
17.Rafter to plate – toe nail 3-16d
21.Corner studs and angles (built up corners) 16d @ 24"
22.Built-up corners and beams 20d @ 32" at top and bottom staggered, 2-20d at ends and each splice.

REFERENCES
a. Uniform Building Code, 1988 Edition
b. ASTM F-1667-95 Standard Specification for Driven Fasteners: Nails, Spikes, Staples

Figure B-28:
Townhouse Structural Details (Sheet S-2, Left)

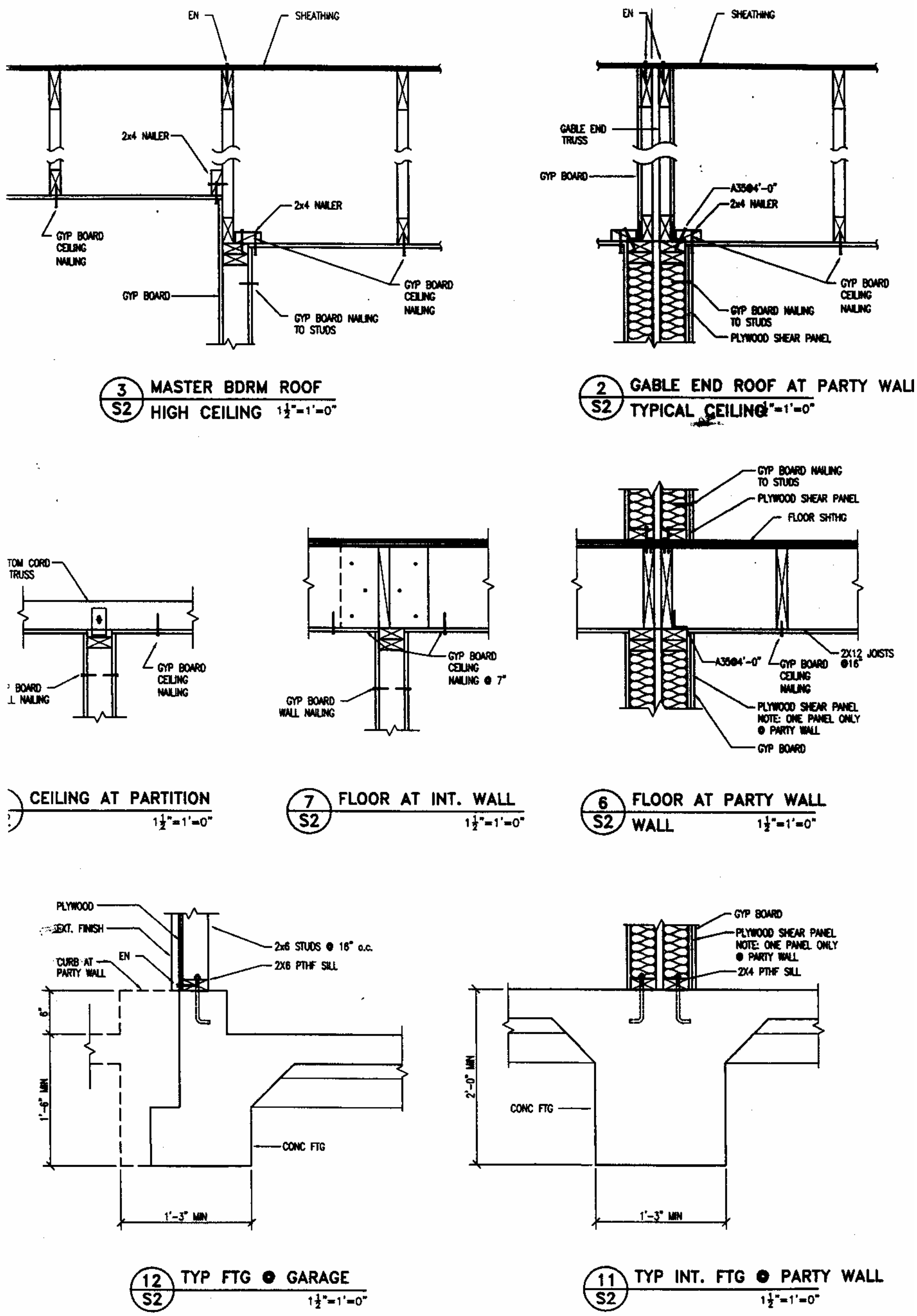


Figure B-29:
Townhouse Structural Details (Sheet S-2, Right)

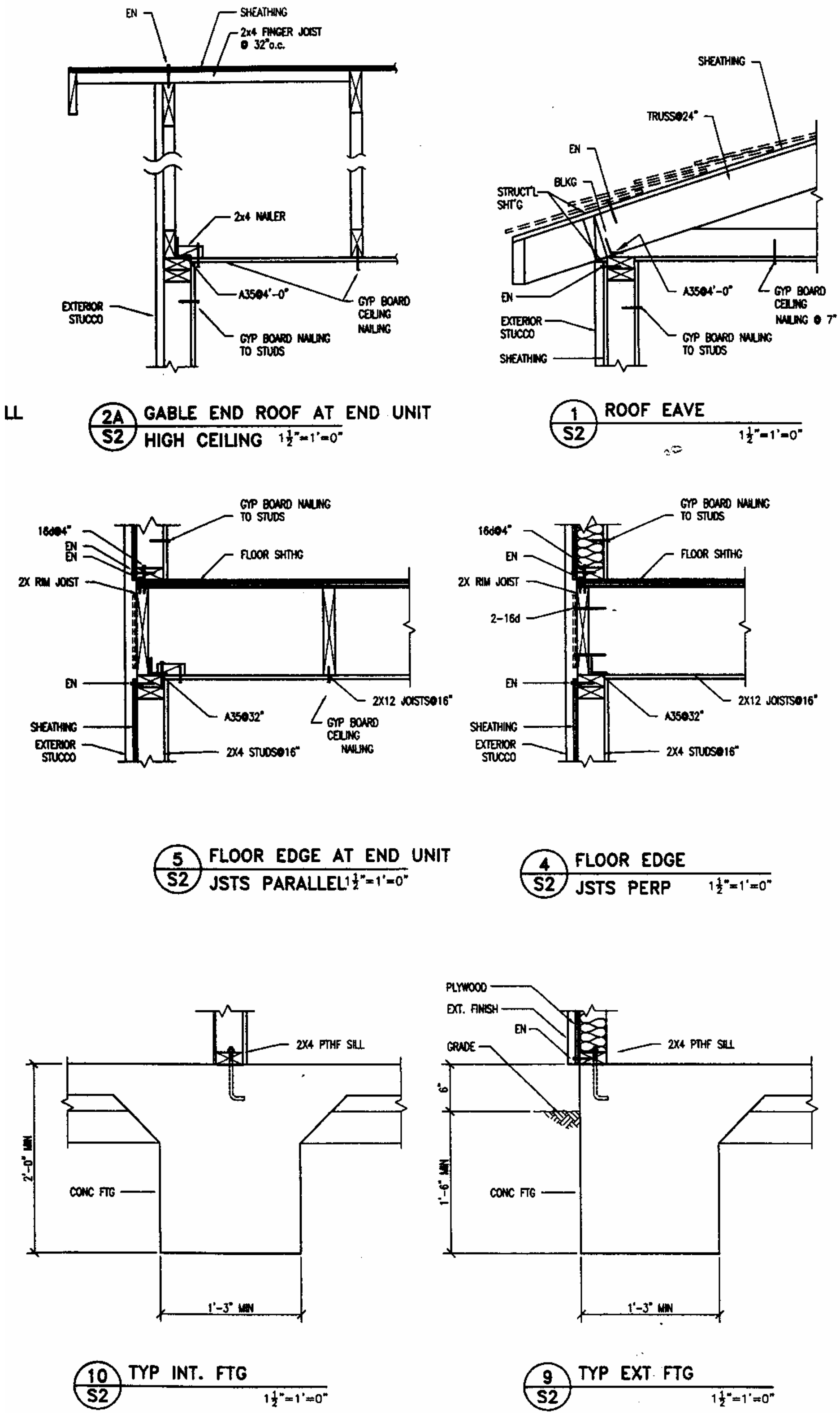


Figure B-30:
Townhouse Structural Plans (Sheet S-3)

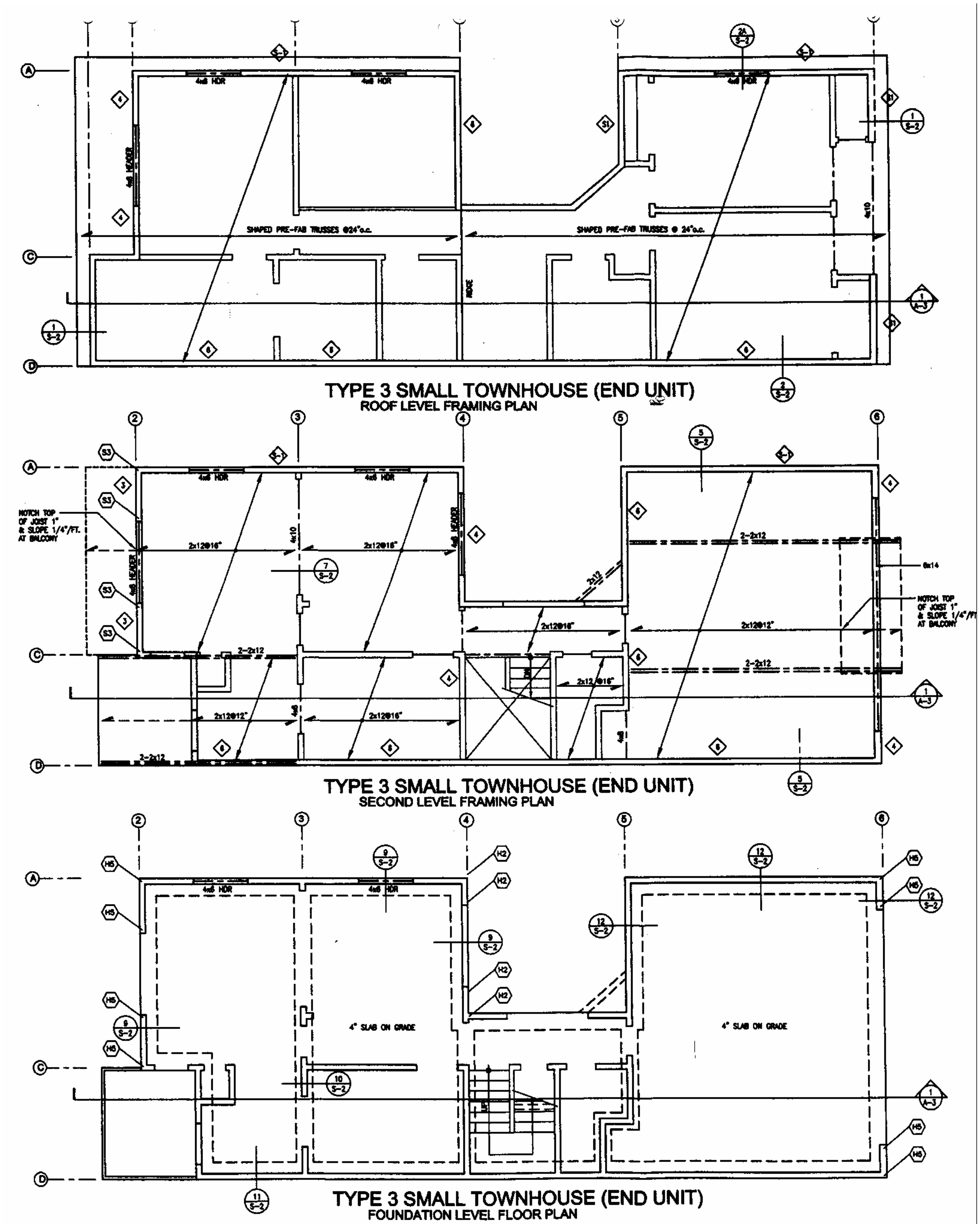


Figure B-31:
Apartment Parking Level Plan (Sheet A-1 Right)

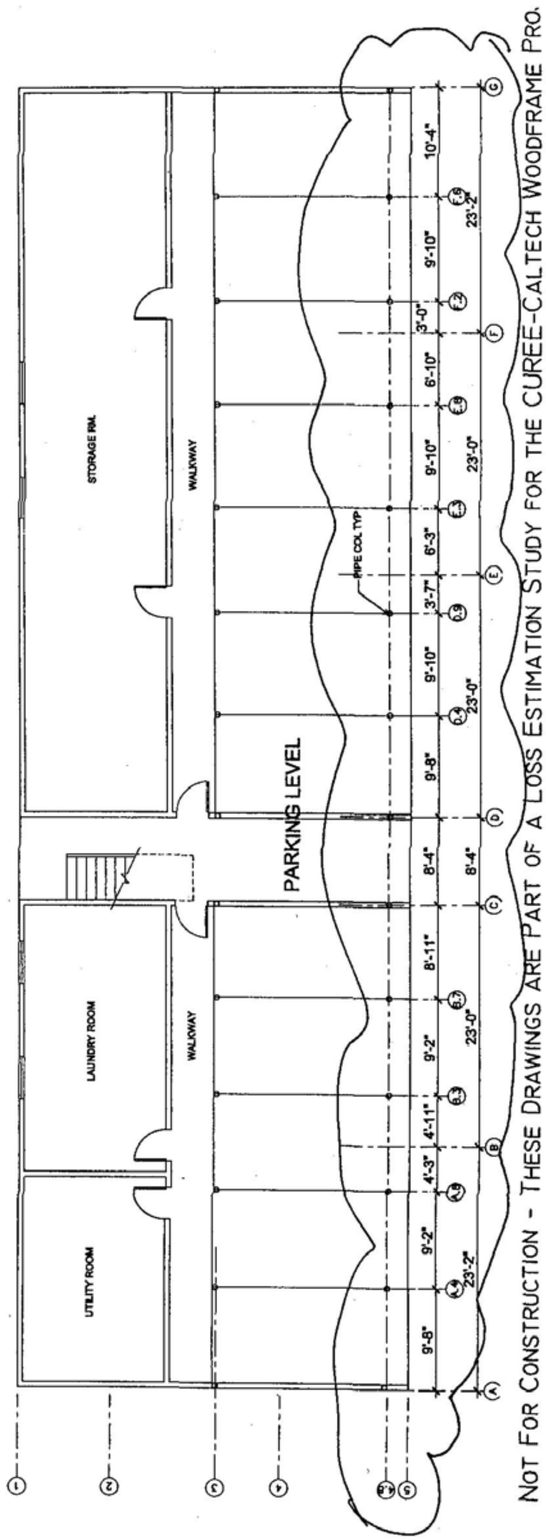


Figure B-32:
Apartment Second And Third Floor Plan (Sheet A-1 Left)

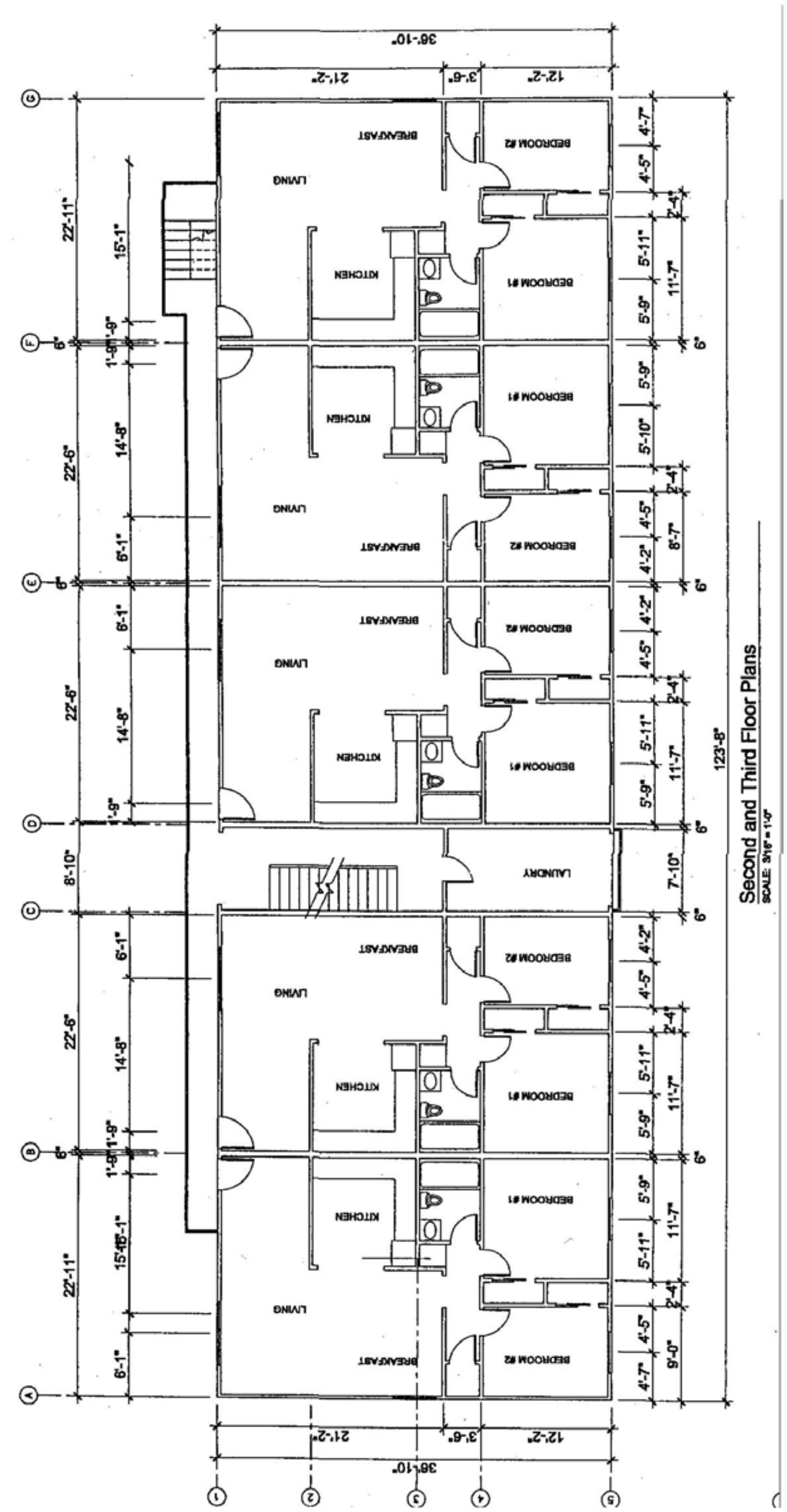


Figure B-33:
Apartment West And South Elevations (Sheet A-2)

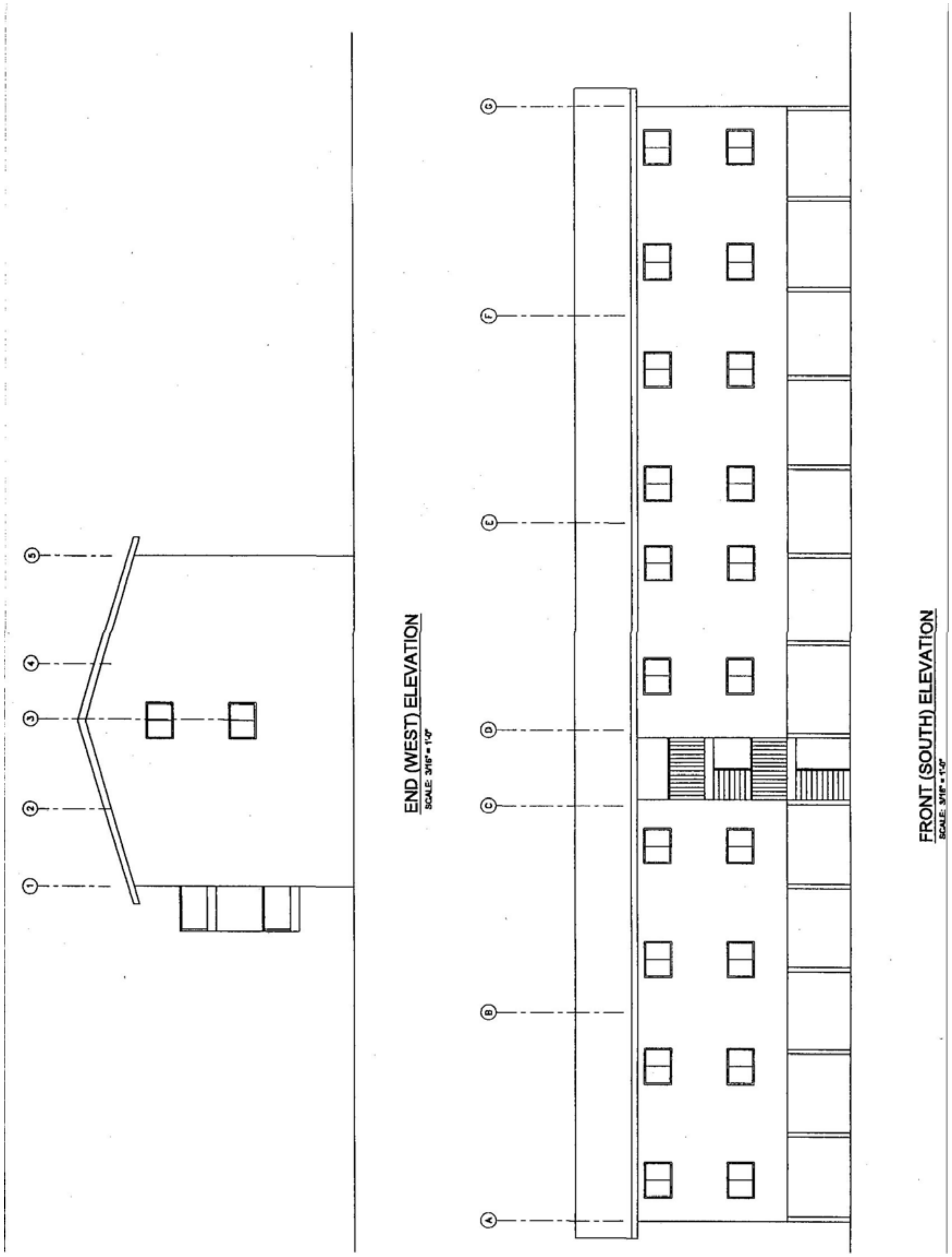


**Figure B-34:
Apartment East And North Elevations (Sheet A-3)**

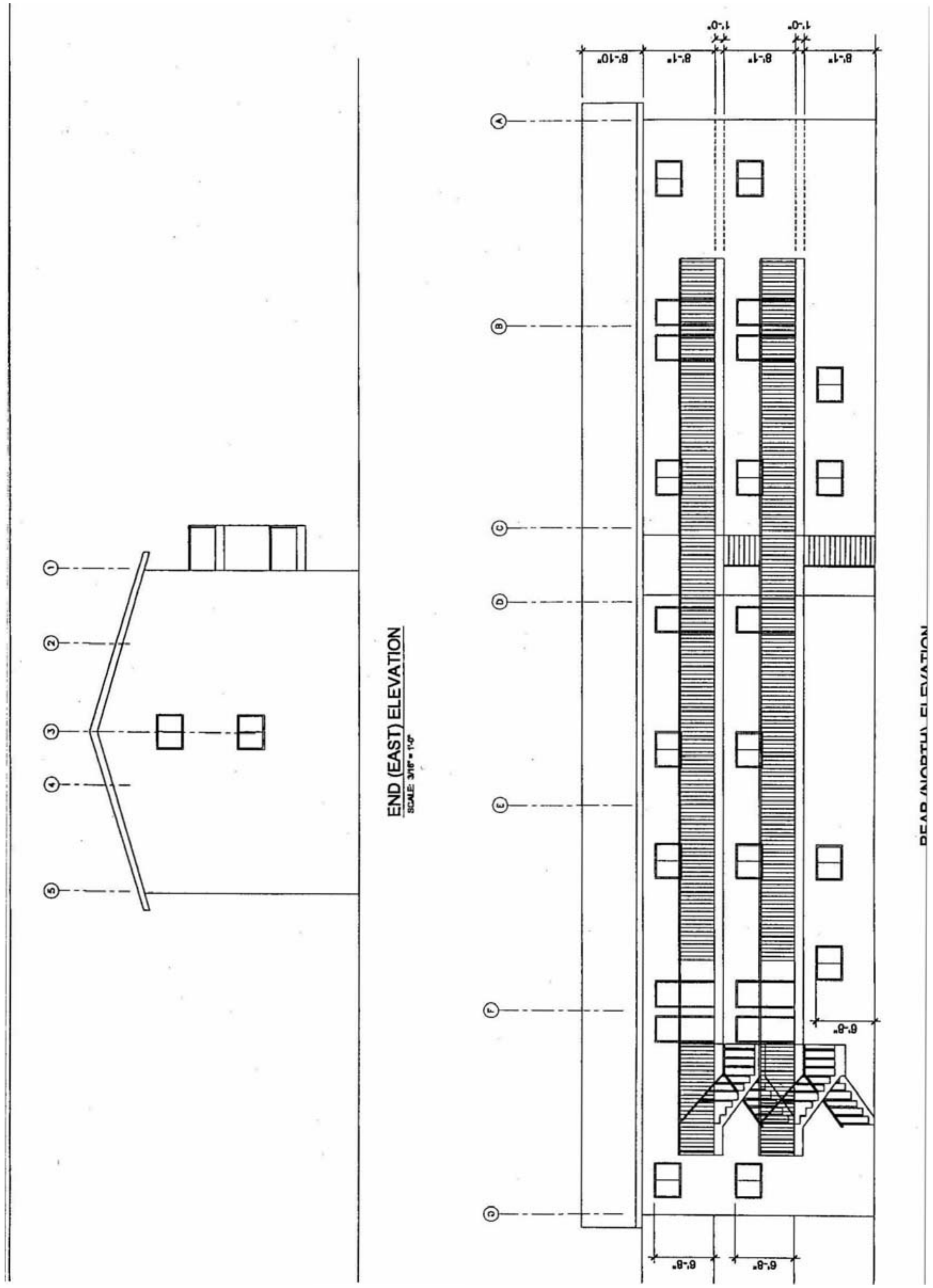
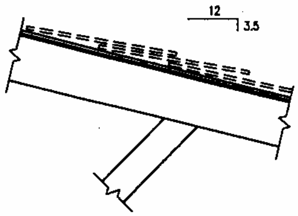


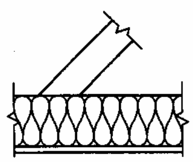
Figure B-35:
Apartment Structural Notes (Sheet S-1, Left)

3.5:12 SLOPED ROOF



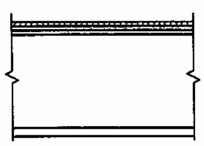
	PSF
COMP SHINGLE	3.5
RE-ROOF	2.0
BUILDING PAPER	
1/2" PLWD	1.5
2X6@24" RAFTER	1.1
2X4@24" STRONGBACK	0.7
SUB-TOTAL	8.8
ADJUST FOR SLOPE X1.042	9.2
TOTAL	9.2

CEILING



	PSF
2X6@16" CEILING JOIST	1.7
2X4@24" STRONGBACK	0.7
INSULATION 0.3 #/inX3-1/2	1.1
FINISH MIN 5/8" GYP	2.3
TOTAL	5.8

FLOOR



	PSF
FLOORING	1.0
5/8"T&G PLWD	1.8
2X12@16"	3.1
5/8" GYPBOARD	2.3
TOTAL	8.2

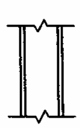
(41 PSF EXTERIOR BALCONY W/ 2" CONC TOPPING & STUCCO SOFFIT)

EXTERIOR WALL

	PSF
7/8" 3 COAT STUCCO	8.8
3/8" PLWD	1.1
2X4 STUDS@16"	1.1
TOP PLATES	0.3
INT FINISH 1/2" OR 5/8" GYP	2.3
TOTAL	13.6

1/4" GLAZING + FRAME ~ 4.0 PSF

INTERIOR WALL



	PSF
2X4 STUDS@16"	1.1
TOP PLATES	0.3
FINISH - 2 X 1/2" OR 5/8" GYPBD	4.6
TOTAL	6.0

WOOD SHEAR WALL SCHEDULE

SYMBOL	ALLOWABLE SHEAR (PLF)	SHEATHING MATERIAL	MIN STUD AT ADJOINING PANEL EDGES	FOUNDATION SILL	SHEATHING EDGE NAILS (1)	SHEATHING INTERMEDIATE NAILS (1)	ANCHOR BOLTS (2)
6	270	3/8" STR I PLWD	2x	2x	8d@6"	8d@12"	NA
4	398	↓	2x	2x	8d@4"	↓	3/4"@32"
3	450	↓	2x	2x	8d@3"	↓	3/4"@32"
G7	100	1/2" GYPBOARD UNBLOCKED	2x	2x	(3) 5d@7"	(3) 5d@7"	1/2"@6'-0"

1. Common nails, substitutions must be approved by Engineer.
2. Minimum two bolts per piece of sill.
3. Cooler nails.

Figure B-36: Apartment Structural Notes (Sheet S-1, Right)

The Apartment Index Building is one of four index buildings prepared for Element 4 – Economic Aspects, as part of the CUREe–Caltech Woodframe Project. This building is being used as part of structural analysis studies conducted under Task 1.5.4, which will be used for fragility analyses conducted under Task 4.1.

Drawing sheets A1–A3 and S1–S3 contain information describing the building. Sheets A1–A3 contain the architectural plans, drawn by Ray Young and Associates, based on a description developed by the Woodframe Project. Sheets S1–S3 contain typical details, notes, assembly and weight information, provided by Element 3.

The apartment is a three story multi–family dwelling. It is intended to have been built in the 1960's, located in either Northern or Southern California. The design is based on partially engineered construction. In particular, the unit shears in the plywood shear walls have been checked in accordance with the 1964 UBC. To the extent possible, characteristic materials and fastening have been identified.

SPECIES
Typical species for framing – Douglas–fir.
Foundation sill plates – Foundation grade redwood.

SHEATHING
Roof sheathing 1/2" PLWD.
8d common at 6" supported edges, 12" other supports.
8d common dimensions – ASTM F1667:
Flat head, diamond point, L=2.5", D=.131"

Floor sheathing 5/8" T&G PLWD
10d common at 6" / 12".

Wall sheathing 3/8" STR I PLWD.
8d common. See shear wall schedule for edge nail
spacing. 12" field spacing.

Gypsum wallboard sheathing – ½" sheathing – probably applied horizontally.
1988 UBC describes cooler nails. Per ASTM F1667 these are:
Smooth shank––Flat–head diamond point 5d – 13–1/2 gage, 1–5/8 long

For gypsum board wall sheathing one of these fasteners are at 7 inches on center over the height of each stud. We understanding is that they would not have been edge–nailed at this spacing to the top or bottom plates. The spacing at top and bottom plates would likely have been 16 inches on center as part of the vertical line of fasteners at each stud.

Gypsum board ceiling sheathing. The fasteners above are at 7 inches along the ceiling joists. The perimeter edges parallel to the joists would have been nailed in order to provide proper vertical support. The edges perpendicular would not have been nailed.

STUCCO
UBC Tables 47G. 18 ga hexagonal woven wire, furred out from backing ¼" nailed with 3/4" min penetration, spaced 6" maximum vertically, 16" horizontally.

FASTENING

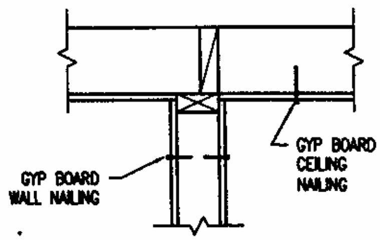
Anchor bolts – per shear wall schedule at noted shear walls, otherwise ½ inch diameter at 6'–0" maximum on center.

Framing nailing was generally done with coated sinker nails. The following is the schedule of minimum fastening from the 1964 UBC. It needs to be kept in mind that much of the framing nailing was done with 16d sinker gun nails. Fastening noted as having multiple 8d nails were more likely to have one or two 16d sinker nails.

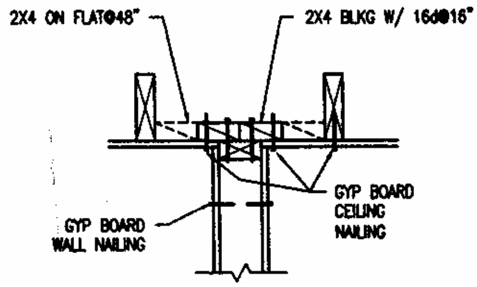
1. Joist to sill or girder – toe nail 2–16d
2. Bridging to joist – toe nail each end 2–8d
6. Sole plate to joist or blocking – face nail 16d @ 16"
8. Stud to sole plate – toe nail 2–16d end or 4–8d toe
10.Doubled top plates – face nail 16d @ 24"
11.Top plates –laps and intersections face nail 2–16d
13.Ceiling joists – to plate – toe nail 2–16d
15.Ceiling joists – laps over partitions face nail 3–16d
16.Ceiling joists to parallel rafters face nail 3–16d
17.Rafter to plate – toe nail 3–16d
21.Corner studs and angles (built up corners) 16d @ 30"

REFERENCES
a. Uniform Building Code, 1961 & 1964 Editions
b. ASTM F–1667–95 Standard Specification for Driven Fasteners: Nails, Spikes, Staples

Figure B-37:
Apartment Structural Details (Sheet S-2, Left)



4/S2 CELINING AT PARTITION PERPENDICULAR 1/2"=1'=0"



3/S2 CEILING AT PARTITION PARALLEL 1 1/2"=1'=0"

Figure B-38:
Apartment Structural Details (Sheet S-2, Right)

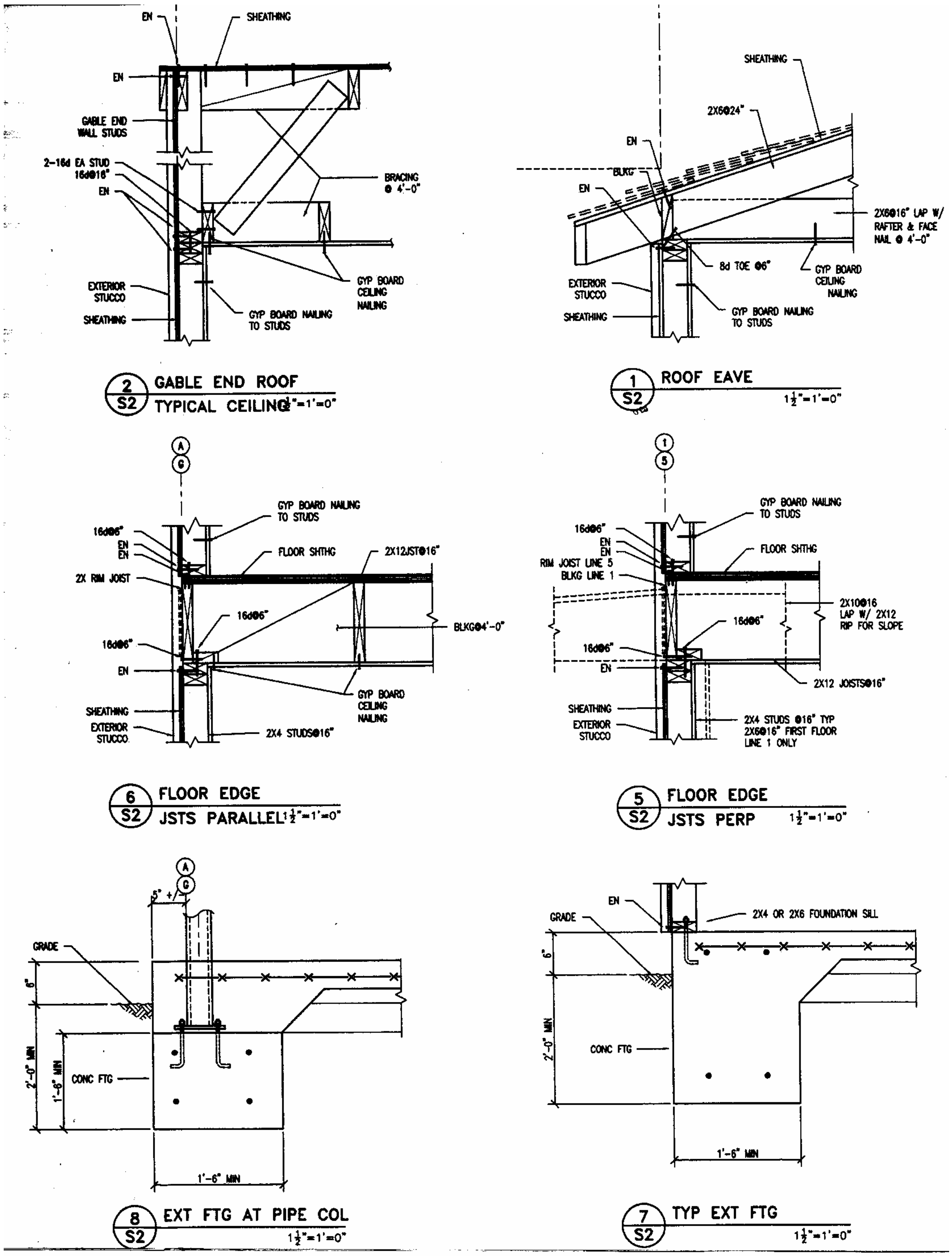


Figure B-39:
Apartment Framing Plans (Sheet S-3, Left)

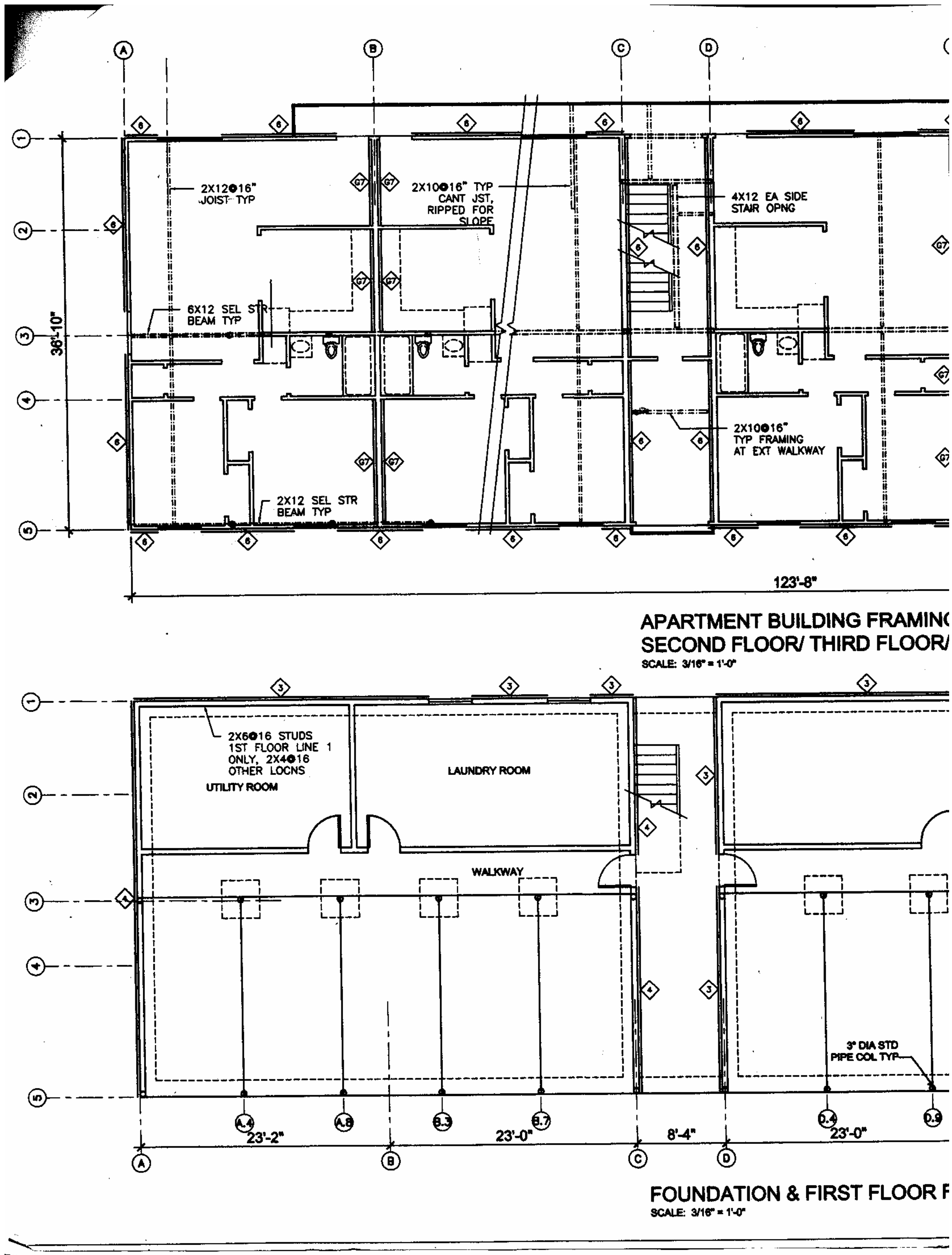


Figure B-40:
Apartment Framing Plans (Sheet S-3, Right)

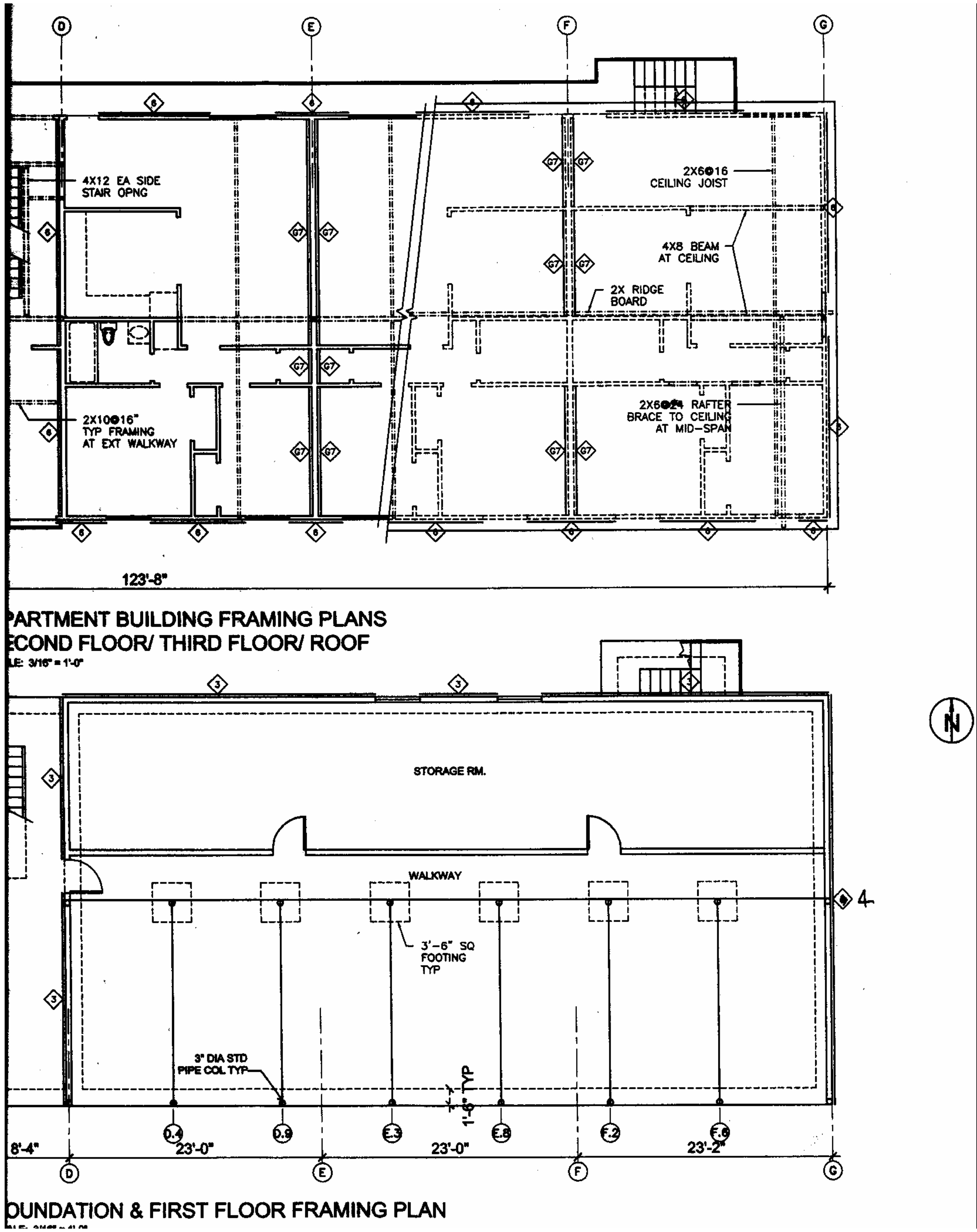


Figure B-41:
Small House Measure 1, Brace Cripple Walls, Plans

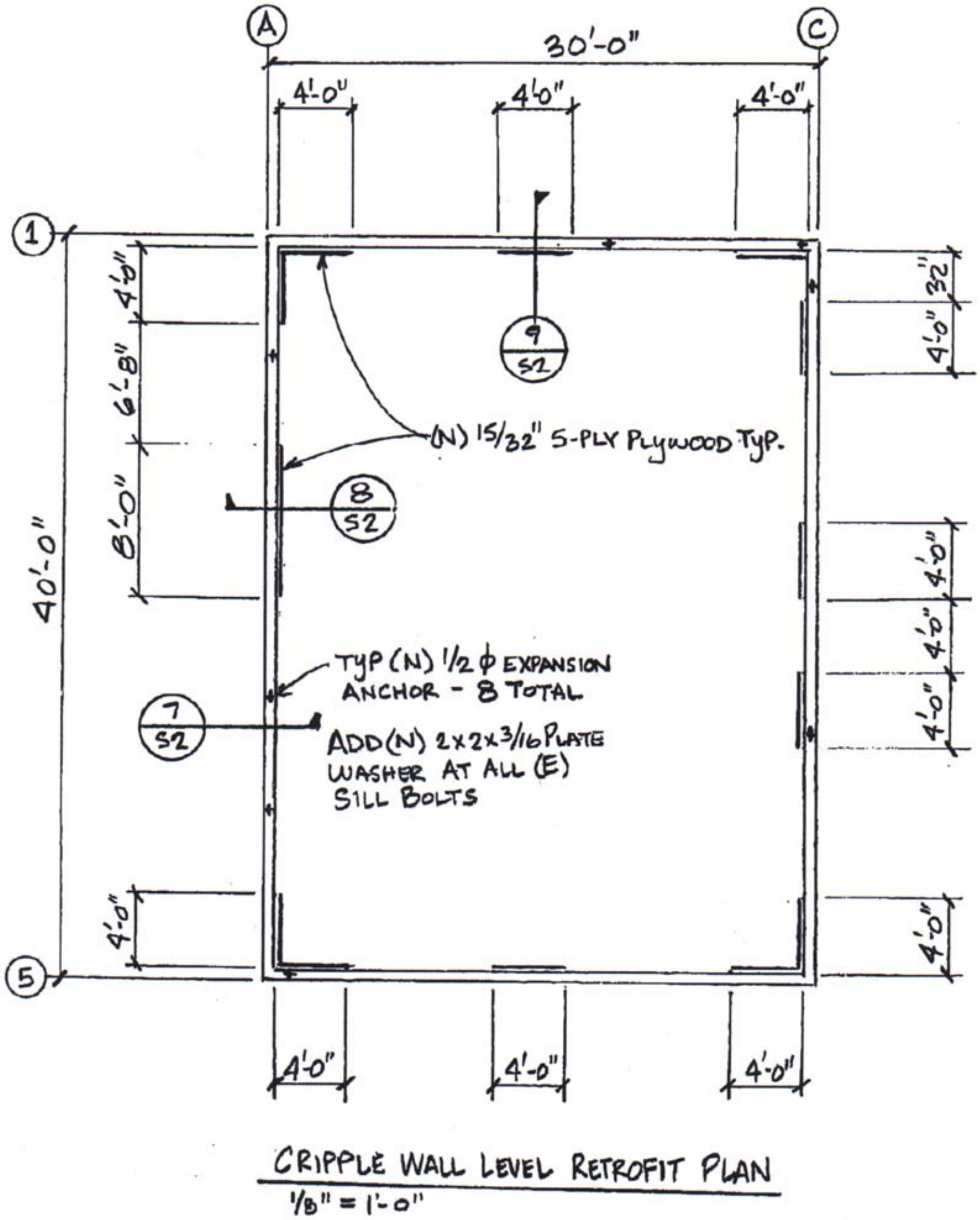


Figure B-42:
Small House Measure 1, Brace Cripple Walls, Details

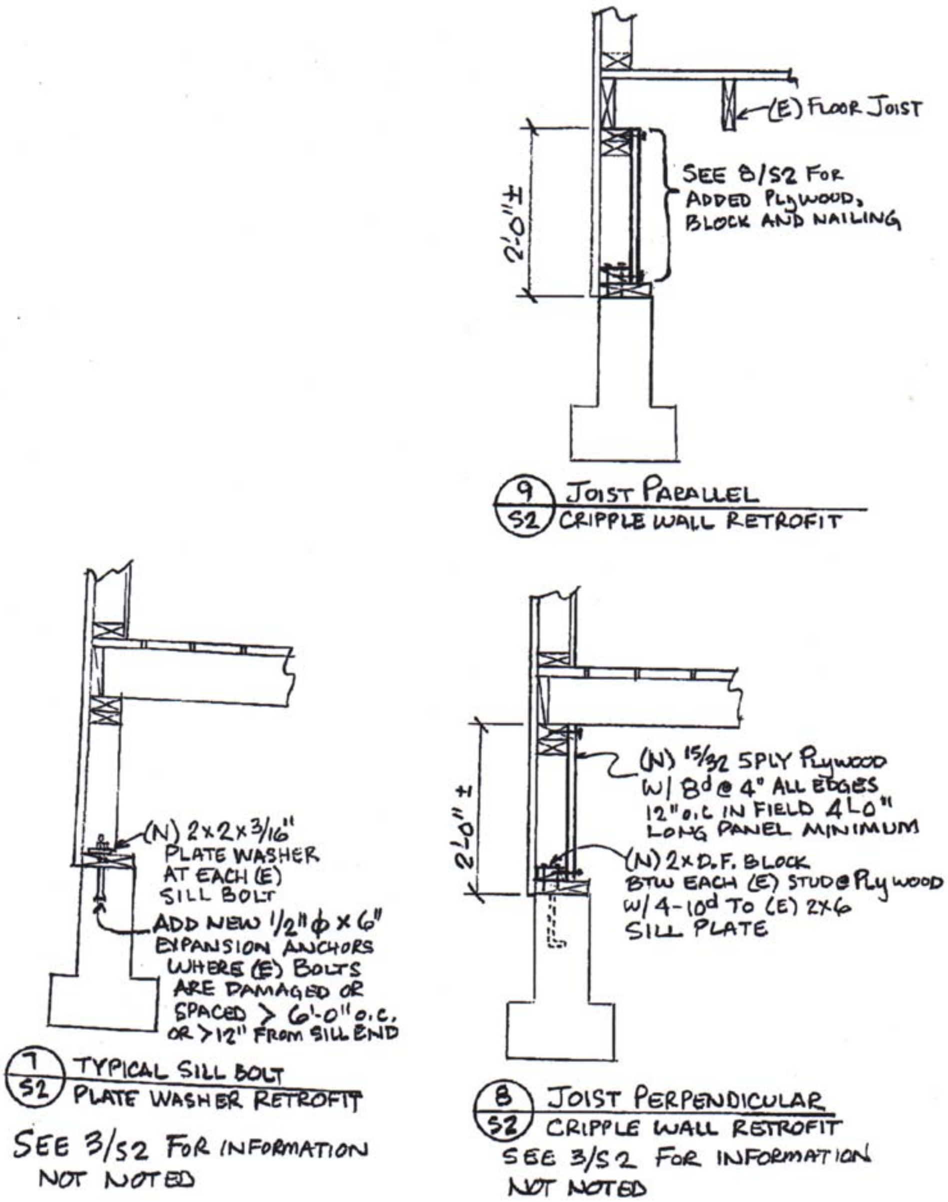


Figure B-43:
Large House Measure 3, Immediate Occupancy First Floor Plans

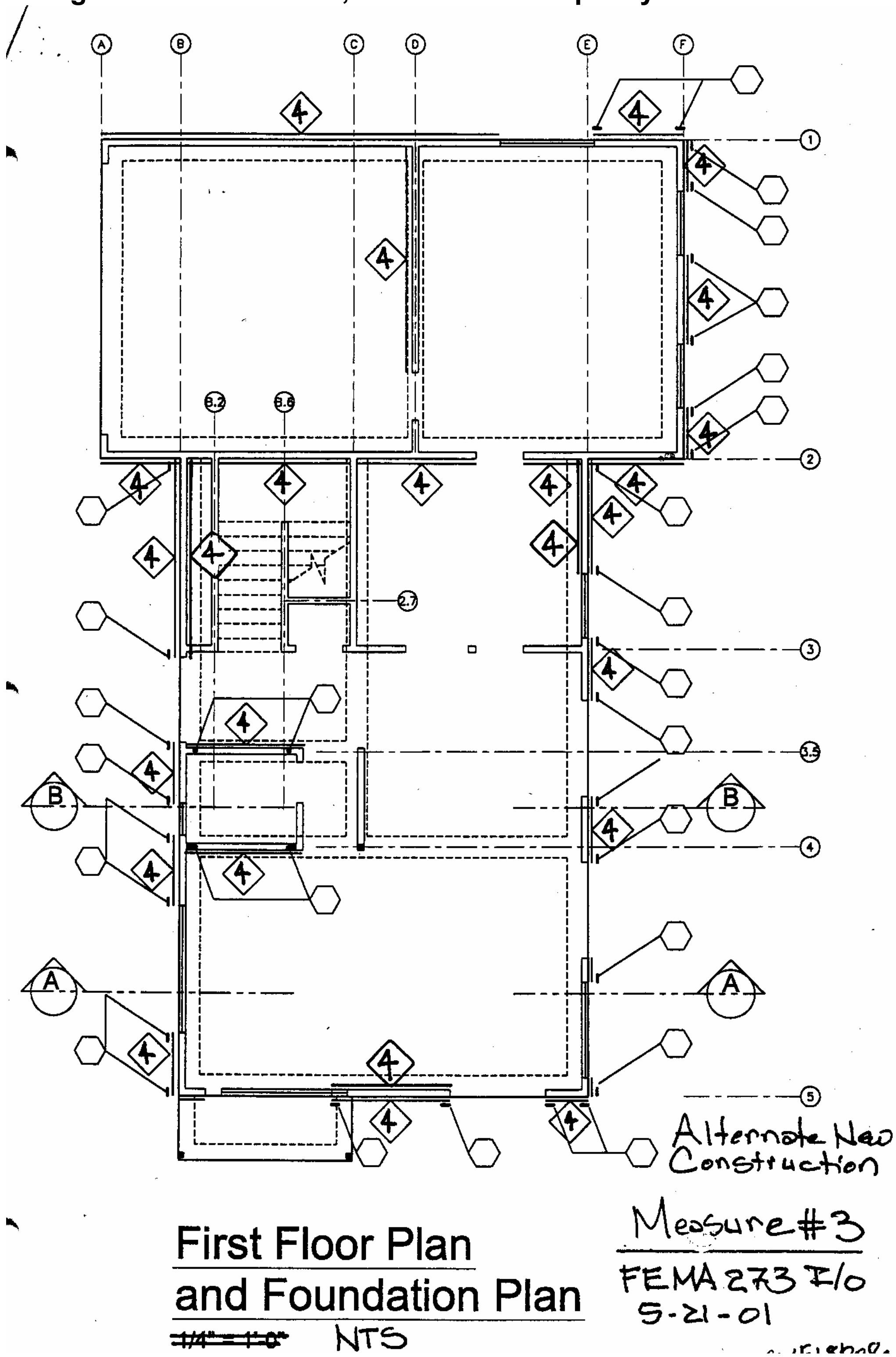


Figure B-44:
Large House Measure 3, Immediate Occupancy Second Floor Plans

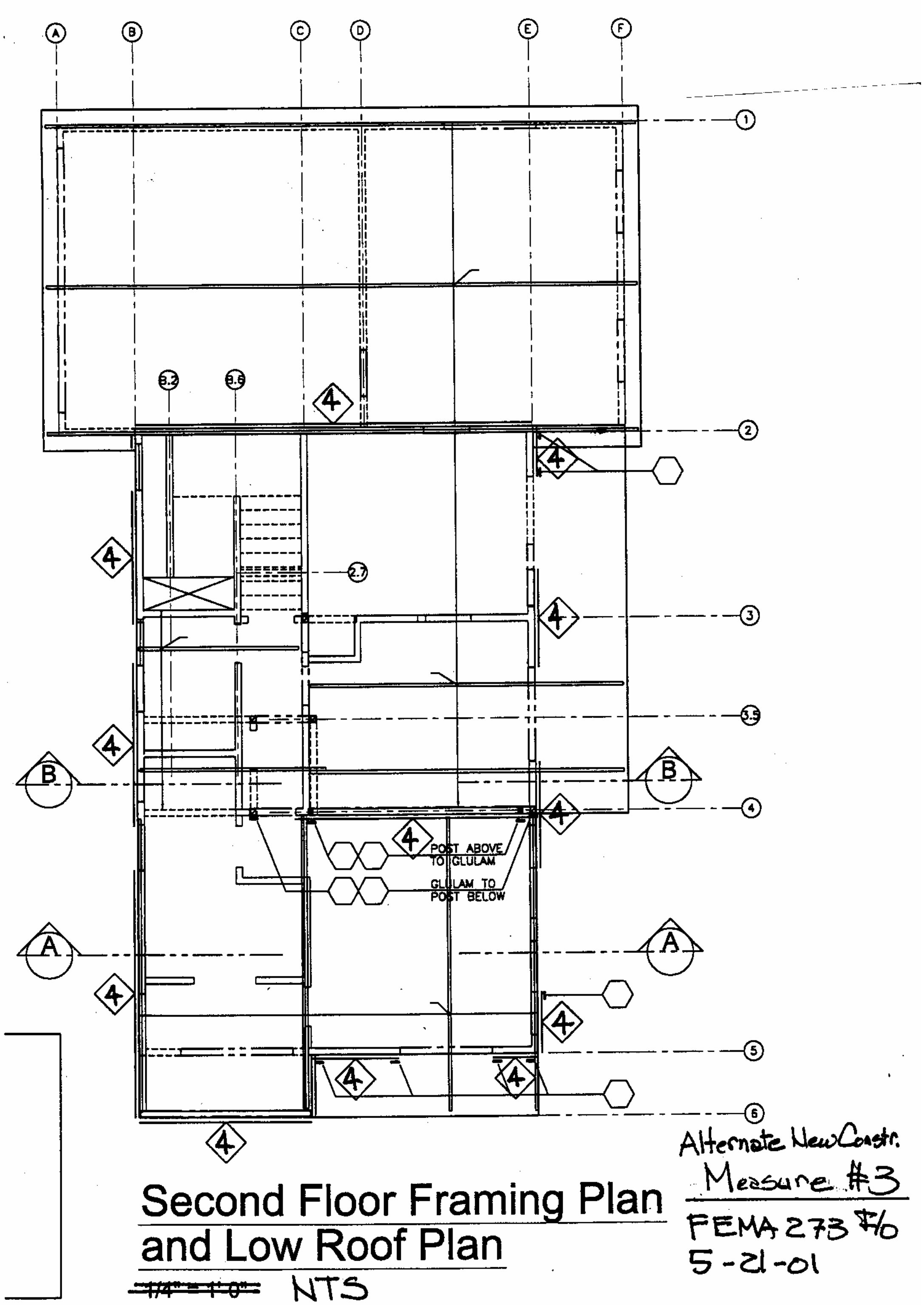


Figure B-45:
Large House Measure 3, Immediate Occupancy Schedules

WOOD SHEAR WALL SCHEDULE							
SYMBOL	ALLOWABLE SHEAR (PLF)	SHEATHING MATERIAL	MIN STUD AT ADJOINING PANEL EDGES	FOUNDATION SILL	SHEATHING EDGE NAILS (1)	SHEATHING INTERMEDIATE NAILS (1)	ANCHOR BOLTS (2)
4	510	15/32 STR I PLWD	3x	3x	10d@4"	10d@12"	TBD

1. Common nails, substitutions must be approved by Engineer.
2. Minimum two bolts per piece of sill.

TIEDOWN SCHEDULE			
SYMBOL	TIEDOWN	ALLOWABLE UPLIFT (#)	ADJUSTED* ALLOWABLE UPLIFT (#)
P18	PA18	2155	NA
P28	PA28	3685	NA
H2	HD2	3265	NA
H5	HD5	4385	NA
S1	ST22	1215	550
S2	ST6224	1875	940
S3	ST6236	2475	1650
S4	MST48	3345	2510

* ADJUSTMENT ONLY CONSIDERS NAILS FALLING IN POST ABOVE & BELOW. NAILS AT FLOOR DEPTH ARE DEDUCTED.

Alternate New Construction Measure #3 FEMA 273 I/O 5-23-01

Figure B-46:
Large House Measure 4, Rigid Diaphragm First Floor Plans

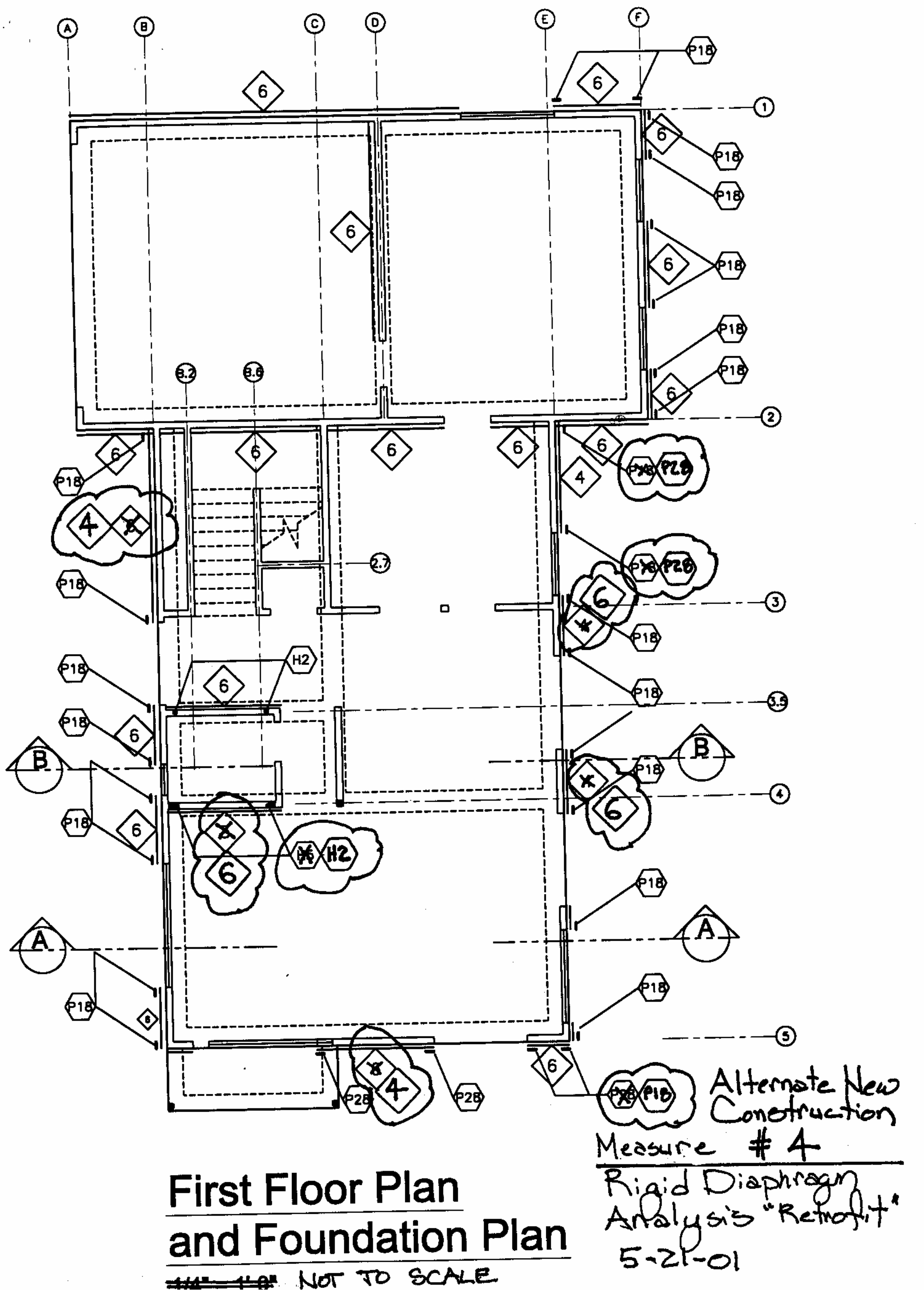


Figure B-47:
Townhouse, Measure 5, Limited Drift, First Floor Plan

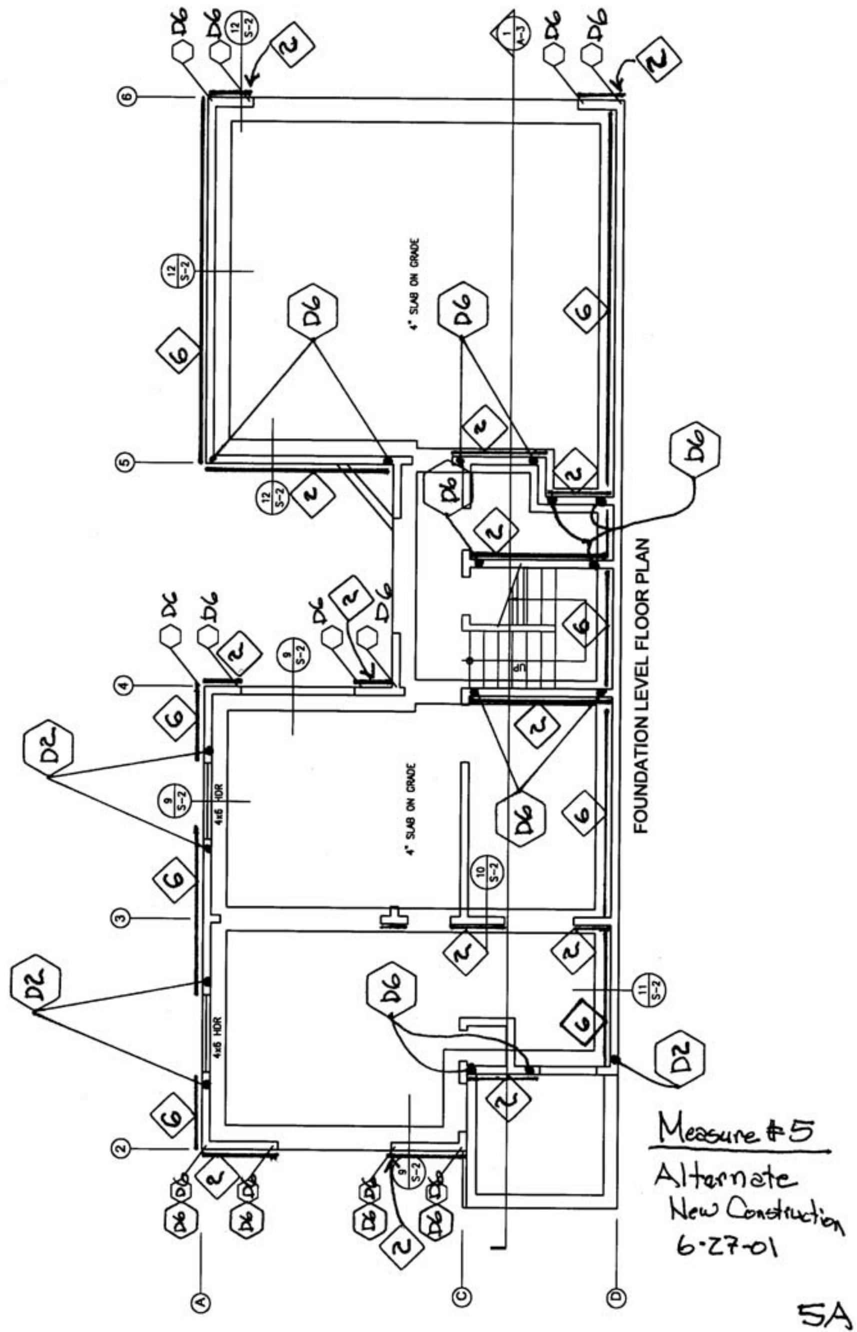


Figure B-48:
Townhouse Measure 5, Limited Drift, Shearwall Schedule

SHEAR WALL SCHEDULE

SYMBOL	ALLOWABLE SHEAR (PLF)	SHEATHING MATERIAL	MIN STUD AT ADJOINING PANEL EDGES	FOUNDATION SILL	SHEATHING EDGE NAILS (1)	SHEATHING INTERMEDIATE NAILS (1)	ANCHOR BOLTS (2)	
6		15/32 STR I Rated Plwd Sheathing	2 x	3 x	8d@6"	8d@12"	~~1/2"@48"~~	5/8@ 48"
4								
3								
2			3 x	3 x	8d@2"		~~1/2"@16"~~	5/8@ 16"
S1								
S2								
G1								

1. Common nails, substitutions must be approved by Engineer.
2. Minimum two bolts per piece of sill.

Tie Down Schedule

D2 = PHD2 on 4x4 DF #2 POST

D6 = PHD6 on 4x4 DF #2 POST

USE 4x6 # 1 post where 2·D6 are noted.

Measure #5
Alternate
New Construction

Figure B-49:
Apartment Building Measure 7, Steel Moment Frames, First Floor Plan

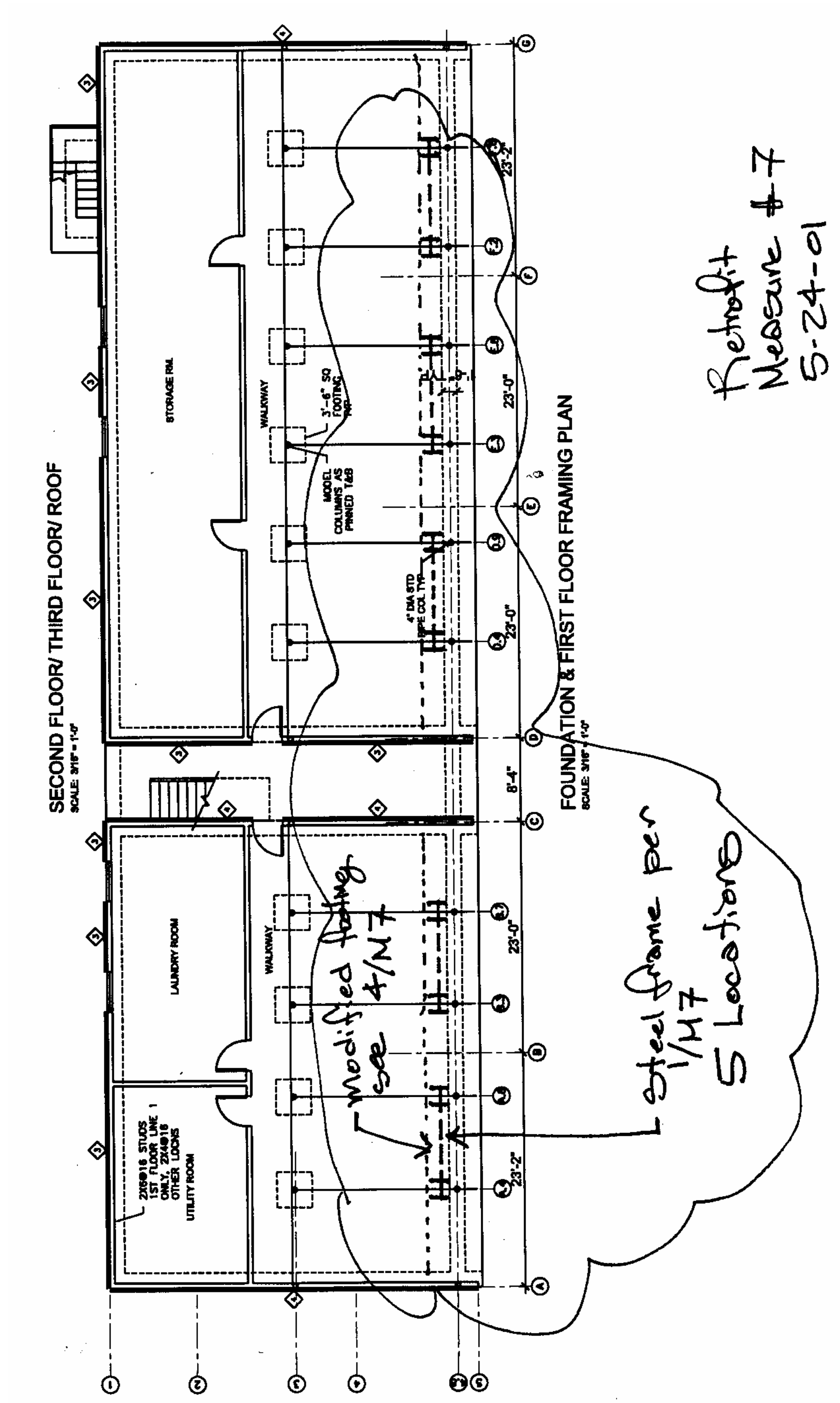


Figure B-50:
Apartment Building Measure 7, Steel Moment Frames, Detail 1

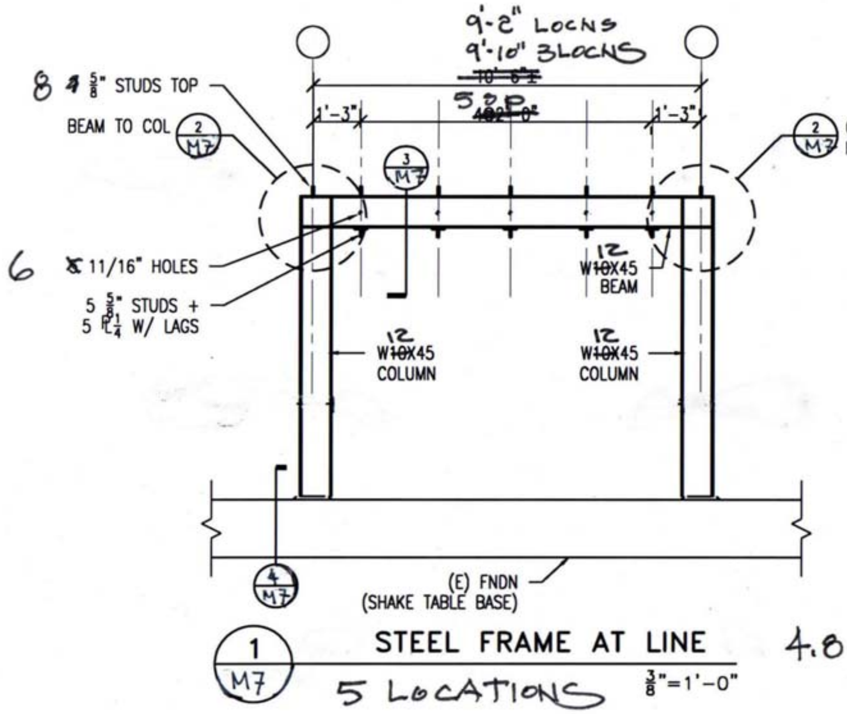


Figure B-51:
Apartment Building Measure 7, Steel Moment Frames, Detail 2

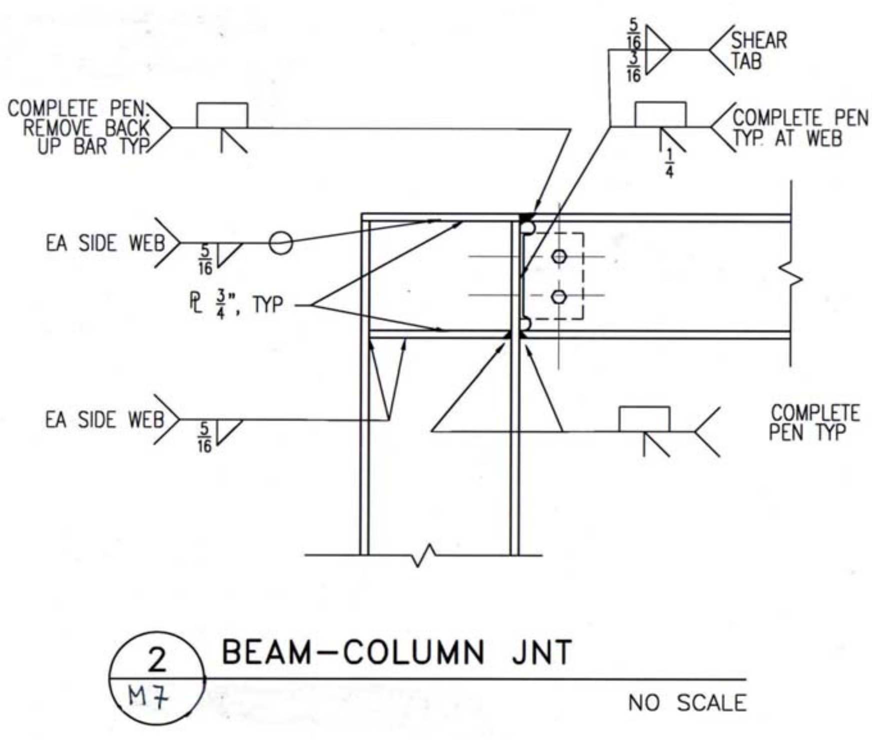


Figure B-52:
Apartment Building Measure 7, Steel Moment Frames, Detail 3

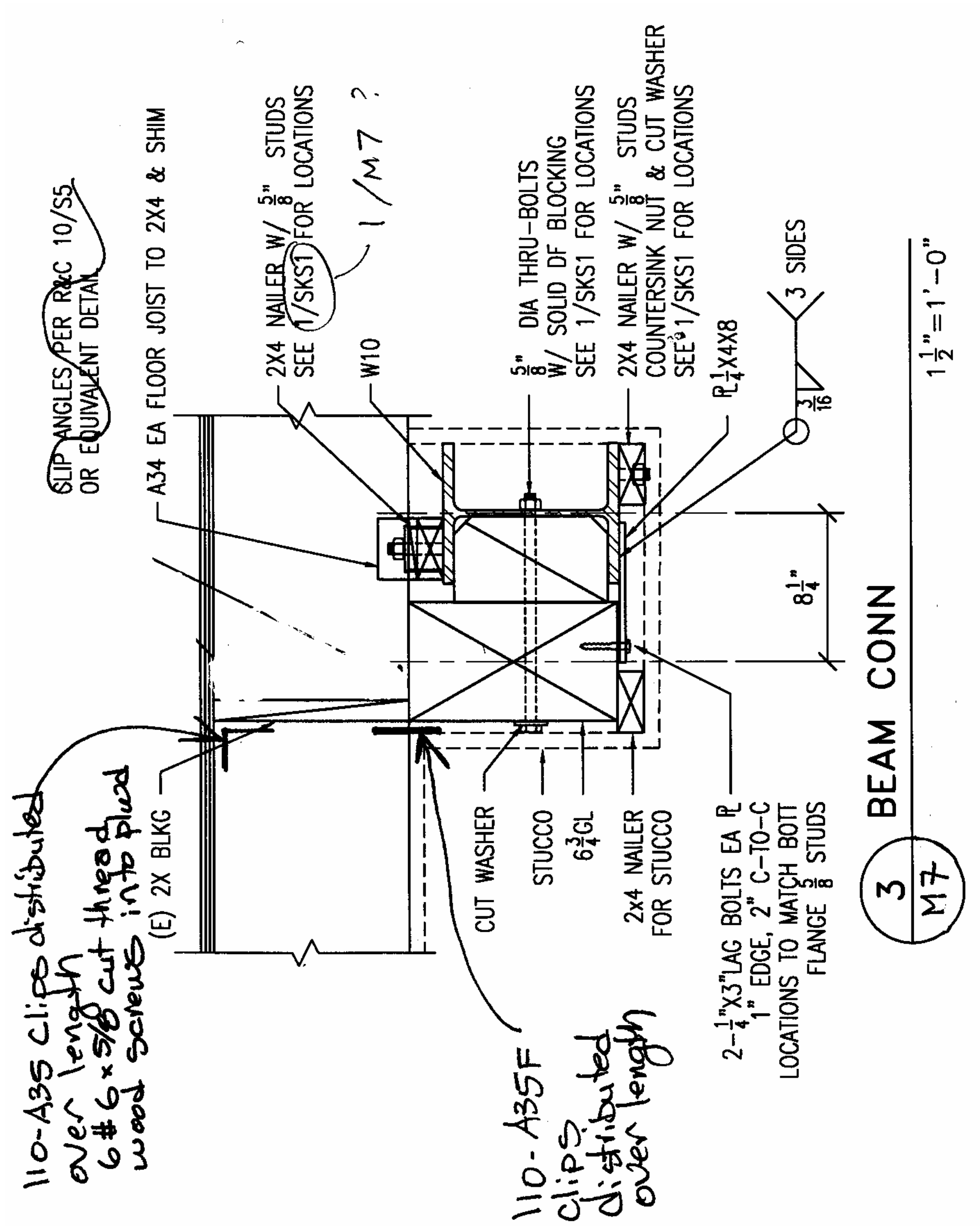


Figure B-53:
Apartment Building Measure 7, Steel Moment Frames, Detail 4

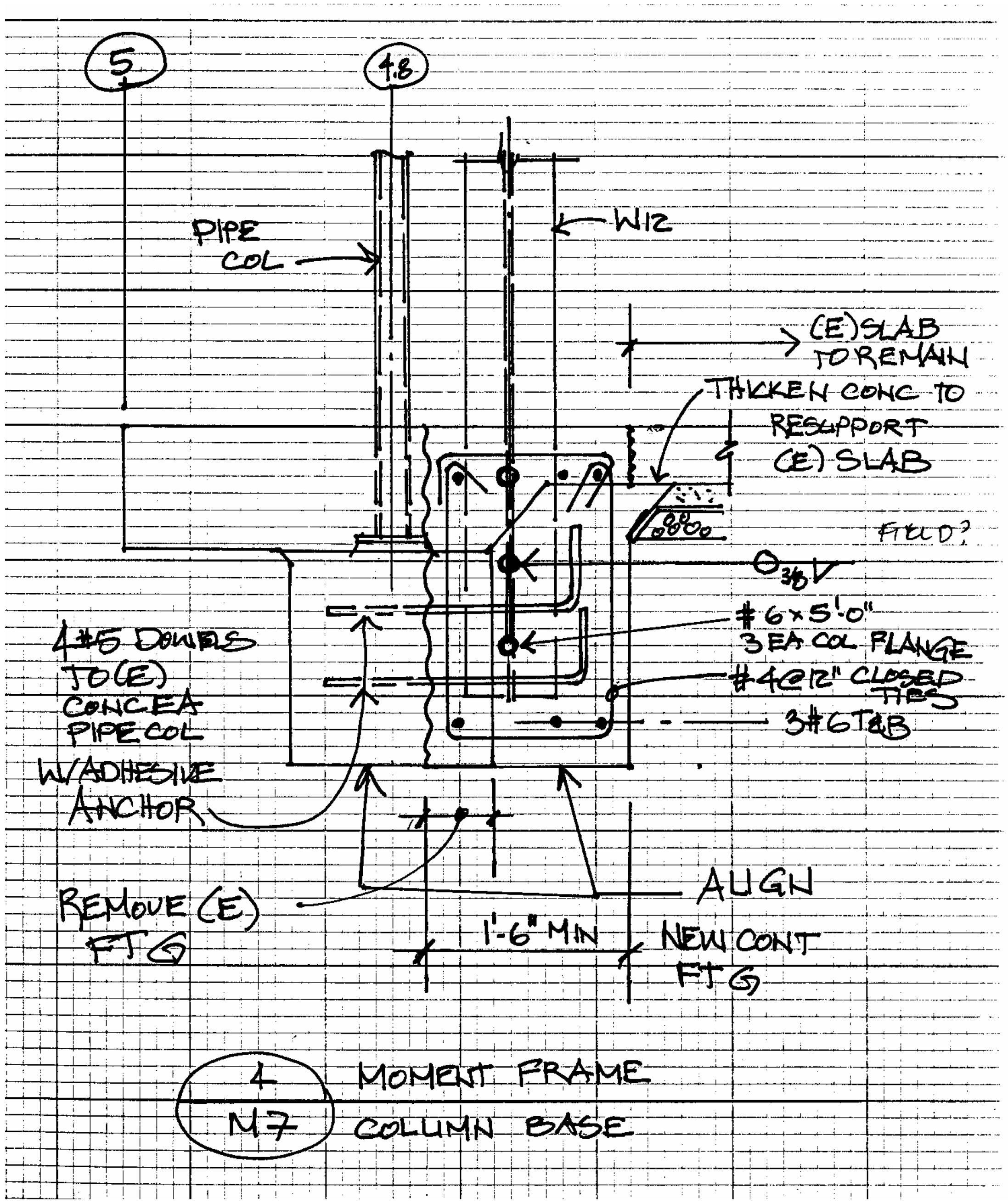


Figure B-54:
Apartment Building Measure 7, Steel Moment Frames, Notes

GENERAL NOTES

STRUCTURAL AND MISCELLANEOUS STEEL

Wide flange beams and their connection plate shall conform to ASTM A572. Grade 50. oR A992

Structural steel shall conform to ASTM A572, Grade 50. Details and workmanship shall be in accordance with the latest AISC Standard Specifications. Welding shall be in accordance with the latest AWS standards using E70XX electrodes.

For complete penetration welds, electrode filler material shall conform to AWS D1.1-94, low hydrogen, providing a minimum Charpy V-notch toughness of 20 foot-pounds at a temperature of zero degrees Farenheit when tested in accordance with AWS A5. Maximum electrode diameter shall conform to AWS D1.1-94 except that electrode diameter for horizontal complete penetration welds shall not exceed 0.125". Following complete penetration welding of the top and bottom flanges, back-up bars and run-off tabs shall be removed and the joint cleaned to allow complete access for weld inspection. Following approval of the complete penetration weld by the Owner's Testing Agency, provide fillet weld as specified in the drawings. Welding procedures the Owner's Testing Agency, provide fillet weld as specified in the drawings. Welding procedures shall be submitted to the Architect and Owner's Testing Agency prior to start of welding.

BOLTS. Bolts shall be ASTM A 307. Bolts bearing on wood shall have standard plate washers under heads and nuts.

SPECIAL INSPECTIONS

The following items shall receive Special Inspection in accordance with UBC Section 1701:

Welding of all structural steel.
Installation of adhesive anchors.

Tension Testing of Adhesive Anchors:

1/2" ø threaded rods 3" into slab:	0 #
1/2" ø threaded rods 16" into slab:	10,000 #

NONDESTRUCTIVE TESTING

Welded, fully restrained connections between the primary members of ordinary moment frames and special moment-resisting frames shall be tested by nondestructive methods for compliance with approved standards and job specifications. This testing shall be a part of the special inspection requirements of UBC Section 1701.5.

The testing program shall include the following:

All complete penetration groove welds contained in joints and splices shall be tested 100 percent either by ultrasonic testing or by radiography.

Any material discontinuities shall be accepted or rejected on the basis of the defect rating in accordance with the (larger reflector) criteria of approved national standards.

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GENERAL NOTES

Figure B-55:
Apartment Building Measure 8, Shearwalls, First Floor Plan

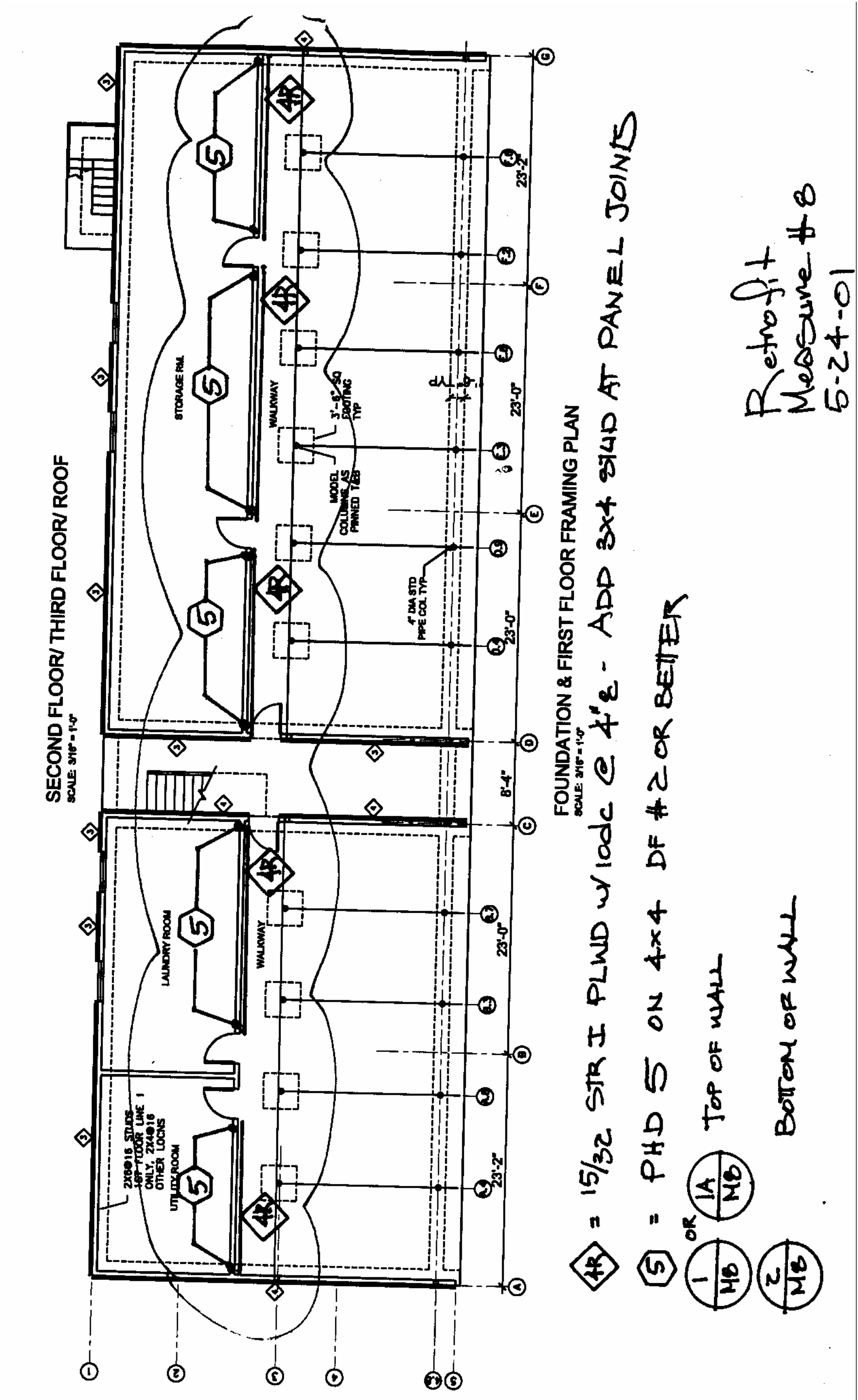


Figure B-56:
Apartment Building Measure 8, Shearwalls, Details 1 and 1A

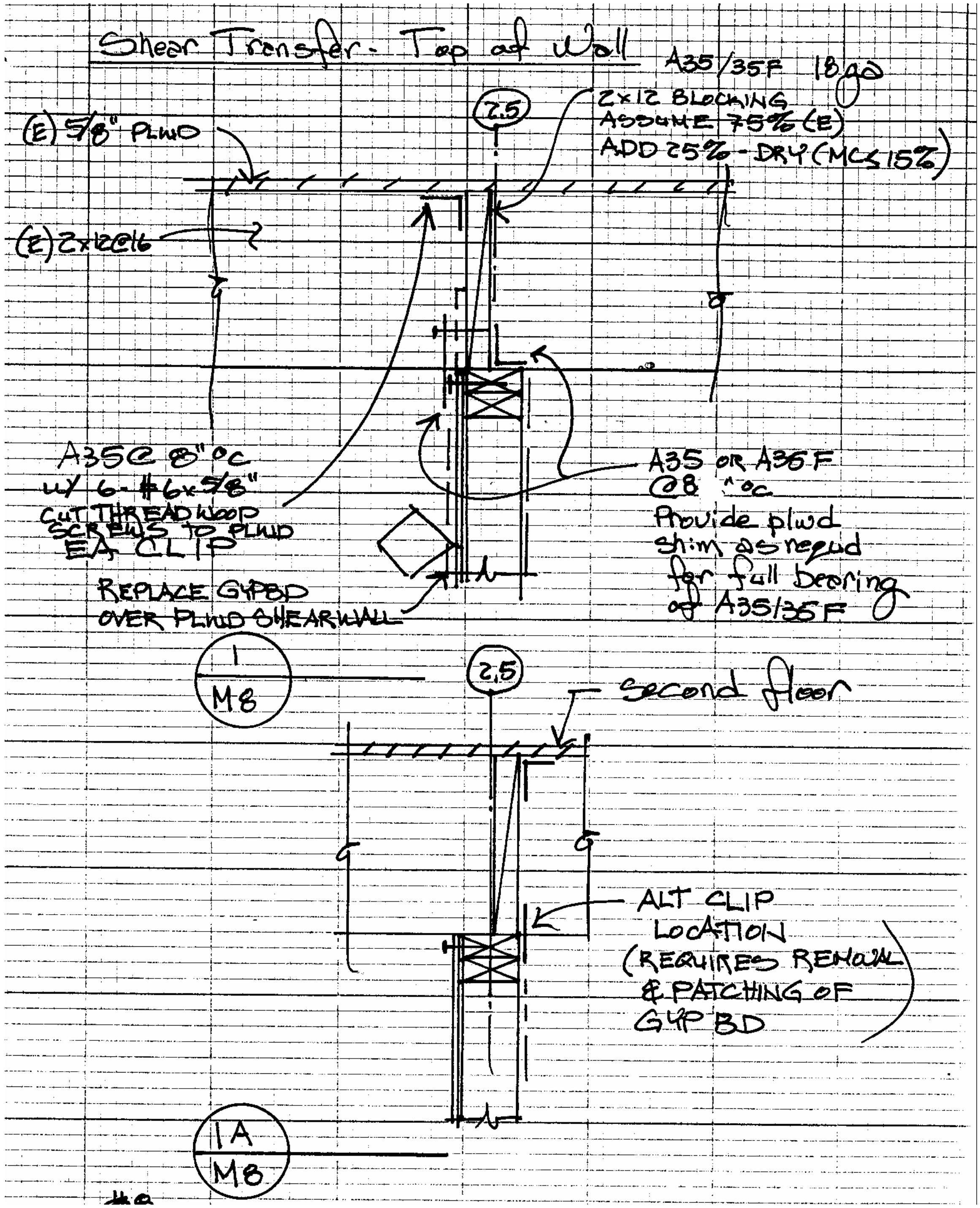
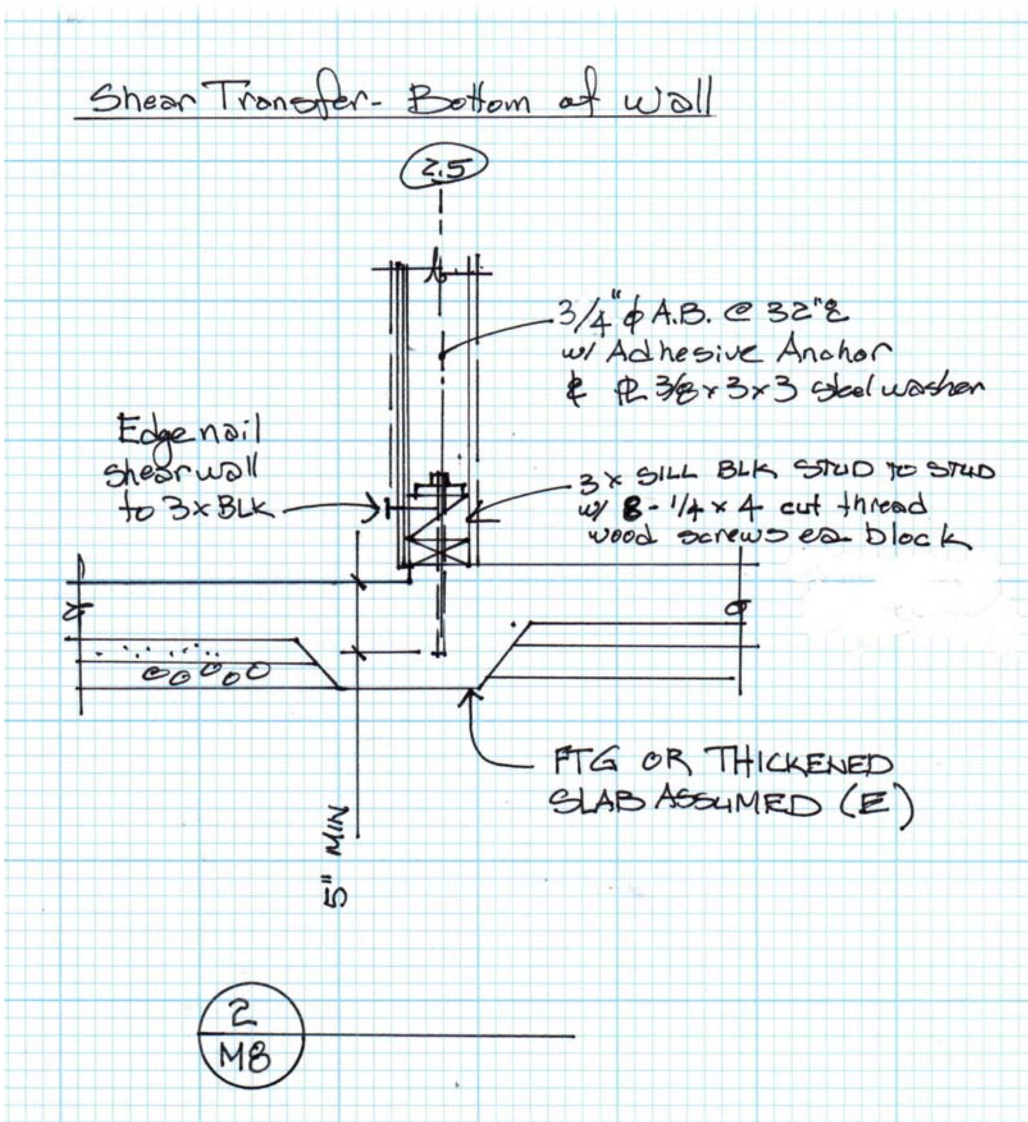


Figure B-57:
Apartment Building Measure 8, Shearwalls, Detail 2



Appendix C. Structural Modeling

STRUCTURAL SYSTEMS

RESEARCH PROJECT

Report No.
SSRP – 2001/12

SEISMIC MODELING OF INDEX WOODFRAME BULDINGS

by

HIROSHI ISODA
BRYAN FOLZ
ANDRÉ FILIATRAULT

Report to the Consortium of Universities for Earthquake Engineering (CUREE) as part of the CUREE-Caltech Woodframe Project ("Earthquake Hazard Mitigation of Woodframe Construction"), under a grant administered by the California Office of Emergency Services and funded by the Federal Emergency Management Agency. Subcontract No. 52: Task 1.5.4 – Analysis of Index Buildings.

September 2001

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University of California, San Diego
Division of Structural Engineering
Structural Systems Research Project

Report No. SSRP- 2001/12

SEISMIC MODELING OF INDEX WOODFRAME BUILDINGS

by

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September 2001

DISCLAIMER

Opinions, findings, conclusions and recommendations expressed in this report are those of the authors. No liability for the information included in this report is assumed by the Consortium of Universities for Research in Earthquake Engineering, the California Institute of Technology, the Federal Emergency Management Agency, or the California Office of Emergency Services.

ACKNOWLEDGEMENTS

The research project described in this report was funded by the Consortium of Universities for Earthquake Engineering (CUREE) as part of the CUREE-Caltech Woodframe Project ("Earthquake Hazard Mitigation of Woodframe Construction"), under a grant administered by the California Office of Emergency Services and funded by the Federal Emergency Management Agency.

We greatly appreciated the input and coordination provided by Professor John Hall and Dr. Keith Porter of the California Institute of Technology and by Mr. Robert Reitherman of the Consortium of Universities for Research in Earthquake Engineering.

The authors acknowledge also Ms. Kelly Cobeen, from GFSD Engineers in San Francisco, who provided construction drawings for the four index buildings and their retrofits modeled in this report.

LIST OF SYMBOLS

A = Cross-sectional area of stem wall

E = Equivalent elastic modulus of horizontal diaphragm

F_i = Force intercept for Wayne Stewart hysteresis rule

F_0 = Force intercept for sheathing-to-framing connector hysteresis rule

F_u = Ultimate lateral capacity of Wayne Stewart hysteresis rule

F_y = Equivalent lateral yield strength of Wayne Stewart hysteresis rule

G = Equivalent shear modulus of horizontal diaphragm

G_c = Shear modulus of concrete

G_d = In-plane stiffness of horizontal diaphragm

k_o = Initial lateral stiffness of Wayne Stewart hysteresis rule

k_s = In-plane lateral stiffness of stem wall

l = Length of stem wall line

PTRI = Post-ultimate tri-linear stiffness factor of Wayne Stewart hysteresis rule

PUNL = Unloading stiffness factor of Wayne Stewart hysteresis rule

r_1 = Asymptotic stiffness factor of sheathing-to-framing connector under monotonic load

r_2 = Post-ultimate strength stiffness factor of sheathing-to-framing connector under monotonic load

r_3 = Unloading stiffness factor of sheathing-to-framing connector hysteresis rule

r_4 = Re-loading pinched stiffness factor of sheathing-to-framing connector hysteresis rule

Rf = Post-yield stiffness factor of Wayne Stewart hysteresis rule

t = Equivalent thickness of horizontal diaphragm

α_1 = Reloading stiffness factor of Wayne Stewart hysteresis rule

β_1 = Softening factor of Wayne Stewart hysteresis rule

Δ_u = Displacement of sheathing-to-framing connector at maximum load

ν = Equivalent Poisson's ratio of horizontal floor diaphragm

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SCOPE OF RESEARCH

The main objective of this research project is to develop numerical models for deterministic nonlinear time-history analyses of four index woodframe buildings under various earthquake ground motions. These index buildings are prototypical buildings developed by the CUREE-Caltech Woodframe Project for use in loss estimation and benefit-to-cost ratio analysis (Porter, 2001). These numerical models will be used for the analyses conducted by the Subcontractor of Task 4.1 (Improving Loss Estimation for Woodframe Building) of the CUREE-Caltech Woodframe project in order to improve the loss estimation methods for woodframe buildings.

The three-dimensional nonlinear seismic analysis model considered is the pancake model that was developed under Task 1.1.1 (Shake Table Test of a Two-Story Single-Family House) of the CUREE-Caltech Woodframe Project (Fischer et al., 2000). The pancake model simulates the three-dimensional seismic response of a woodframe construction through a degenerated two-dimensional planar analysis. The general-purpose computer program RUAUMOKO (Carr, 2001) is used to construct the pancake model of each index building. The required input parameters for wood shear walls, stucco walls, cripple walls and gypsum walls in a three-dimensional woodframe structure (using the Wayne Stewart Hysteric Model, available within RUAUMOKO) is obtained either from the specialized computer platform CASHEW: Cyclic Analysis of wood SHEar Walls, developed as part of Task 1.5.1 of the CUREE-Caltech Woodframe Project (Folz and Filiatrault, 2001), or from available experimental data.

Three RUAUMOKO data files are developed also for three different construction variants (representing superior-, typical- and poor-quality construction) for each of the four basic index buildings. Furthermore, these data files are modified to represent seven different retrofit (or alternate construction) measures for the index buildings. These 19 data files provide a set of hypothetical but realistic woodframe buildings that represent poor, typical and superior construction practices with various retrofit measures.

REPORT LAYOUT

The four index buildings are described in **Chapter 1**. The modeling approach, common to all four index building, is discussed in **Chapter 2 Chapters 3 to 6** describe in details the numerical model of each index building. General conclusions of the project are provided in **Chapter 7**. A list of references is included in **Chapter 8**. The architectural and structural drawings of each of the four index buildings considered are provided in **Appendices A to D**.

1. GENERAL DESCRIPTION OF INDEX BUILDINGS

The four index buildings considered in this study are:

1.1 Small House

A one story, two bedrooms, one bath house built circa 1950 with a simple 1200 square foot floor plan on a level lot. Prescriptive construction is assumed. Wood framed floor cripple walls are included in the poor- and typical-quality construction variants.

1.2 Large House

An engineered two story single family dwelling of approximately 2400 square feet on a level lot with a slab on grade and spread footings. This building is assumed to have been built as a housing development “production house” in either the 1980’s or 1990’s.

1.3 Small Townhouse

A two-story 1,500 to 1,800 square foot unit with a common wall. Part of the dwelling is over a two-car garage. It is on a level lot with a slab on grade and spread foundations. It is recently built and the seismic design is engineered.

1.4 Apartment Building

A three story, rectangular building with ten units, each with 800 square feet and space for mechanical and common areas. All walls and elevated floors are woodframe. It has parking on the ground floor. Each unit has two bedrooms; one bath and one parking stall. It would be constructed prior to 1970 and “engineered” to a minimal extent.

2. MODELING APPROACH

Each index building was modeled in the RUAUMOKO program as a planar "pancake" system with the floor diaphragm and roof diaphragm superimposed on top of each other. The pancake model of a simple two-story woodframe building founded on a rigid foundation is shown in Fig. 2.1. In this simple example, the lateral load resisting system is composed of only perimeter shear walls. The foundation of the structural model is connected to the floor diaphragm with four zero-length nonlinear shear springs representing the four first story shearwalls. Similarly, the floor diaphragm is connected to the roof diaphragm with four additional zero-length non-linear shear springs representing the second story shearwalls. The roof and/or ceiling diaphragm, having a high in-plane stiffness in this particular case, is modeled by four plane stress quadrilateral finite elements with very high in-plane stiffness. The floor diaphragm is modeled by 34 plane stress quadrilateral finite elements. The in-plane stiffness of the floor diaphragm is calibrated to simulate the in-plane response of the floor diaphragm.

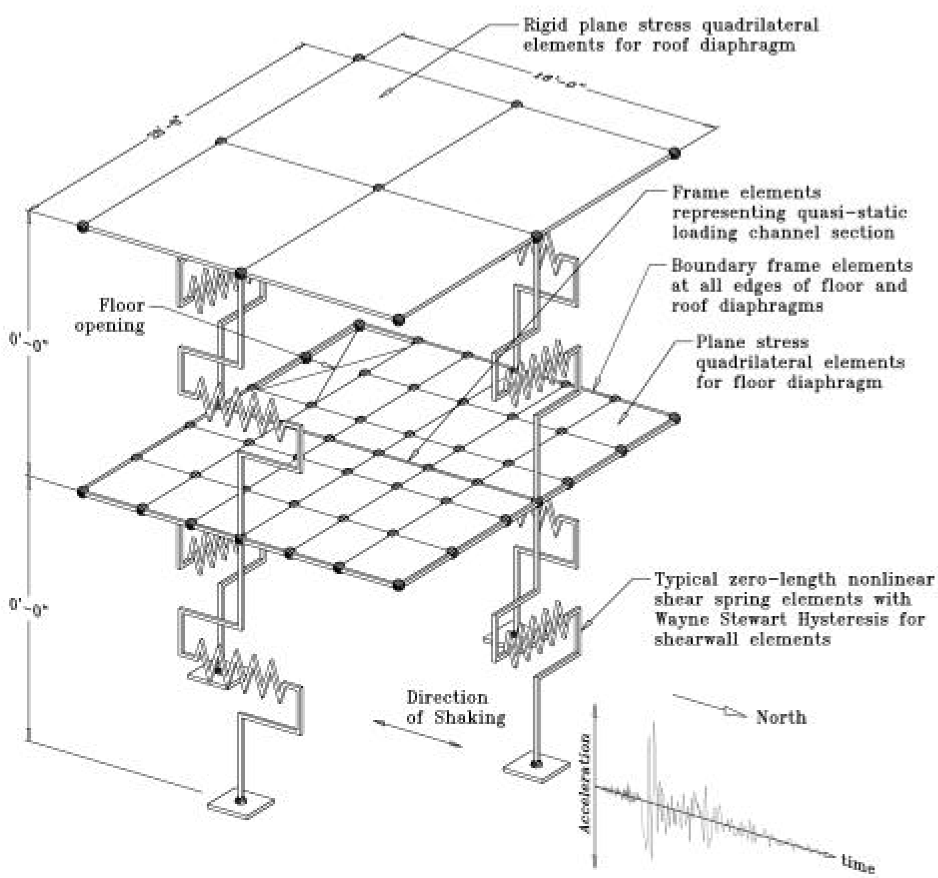


Figure 2.1 Example of a Pancake Model

Frame elements are used along the four edges of the floor diaphragm to connect the corners of the quadrilateral elements to the shear wall elements. The bending stiffness of the frame elements is assumed very small to allow free deformations of the diaphragm. The axial stiffness of the frame elements is assumed very high in order to distribute the in-plane forces of the shear elements along the edges of the floor diaphragm.

Each wall in the structure (shear, cripple and gypsum) is modeled by a single zero-length nonlinear in-plane shear spring using the Wayne Stewart hysteresis rule (Stewart, 1987). Figure 2.2 shows this hysteresis rule and the required input parameters for the RUAUMOKO program. Note that this hysteresis rule incorporates stiffness and strength degradations.

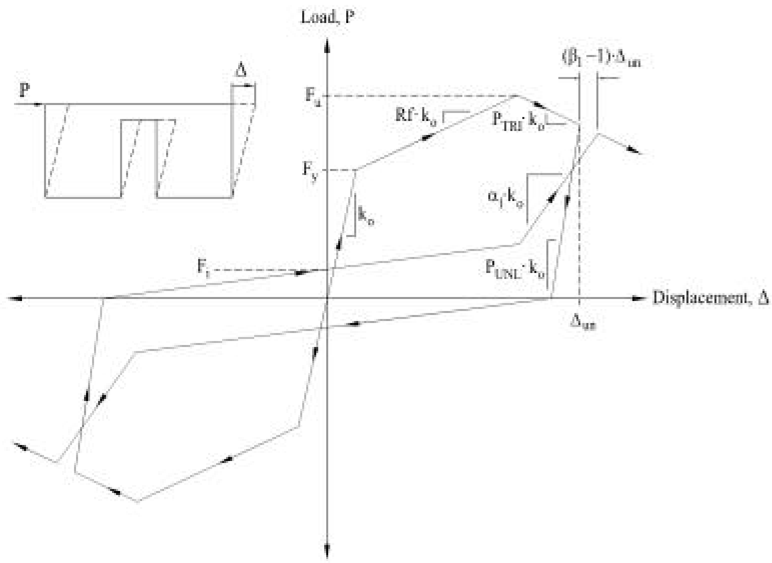


Figure 2.2: Wayne Stewart Degrading Hysteresis Used in RUAUMOKO Program.

The seismic weight of each structure for use in the model is computed using the dead load of all elements. This dead load is distributed as lumped seismic weights at the nodes according to their tributary areas. For each building model, a Rayleigh viscous damping model is considered. This damping model is based on damping ratios of 1% of critical in the first and second elastic modes of vibration. This approach is consistent with the numerical modeling of the two-story woodframe house tested on a shake table as part of Task 1.1.1 of the CUREE-Caltech Woodframe Project (Fischer et al., 2000).

It must be noted that this modeling approach is used to capture the global seismic behavior of the four index buildings. This approach is not intended to model every connection between elements. For example, roof to wall connections or anchor bolts are not considered. Therefore, several characteristics of the various construction variants of each index building are not modeled, as they are believed to be of secondary importance in the global seismic response of the structure.

3. MODELING OF INDEX BUILDING 1: SMALL HOUSE

3.1 General Description

The first index building considered represents a one story, two-bedroom house built circa 1950 with a simple 1200 square foot floor plan on a level lot. Figure 3.1 shows isometric and elevation views of the building. The architectural and structural drawings of the small-house index building are included in Appendix A (Cobeen, 2001).

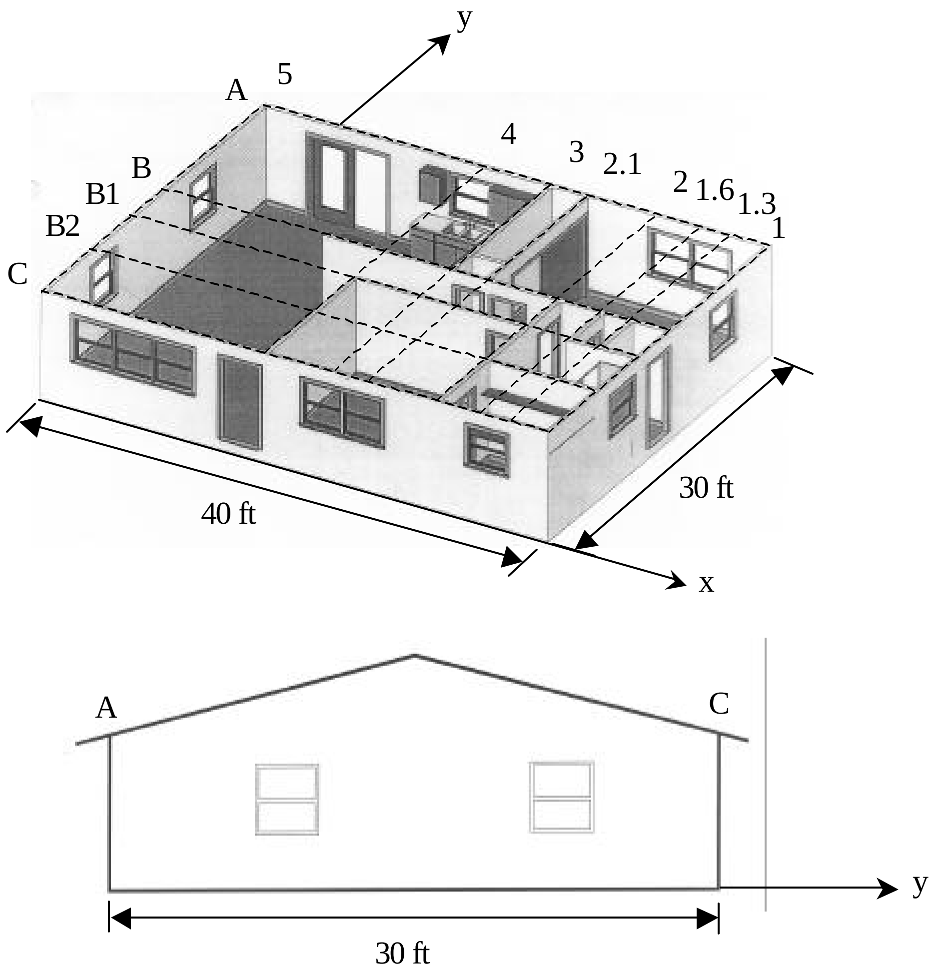


Figure 3.1 Isometric and Elevation Views of Small-House Index Building.

The exterior walls of the small house are sheathed with stucco (3-coat, 7/8-in thick) on the outside and gypsum wallboard (1/2 in thick) on the inside. Furring nails (3/8-in head), spaced at 6 in on center along the vertical studs, are used to attach the wire mesh of the stucco finish to the wood framing. All interior walls are sheathed on both sides with ½ in thick gypsum wallboard. Drywall nails (1-5/8 in long) spaced at 7 in on center along the vertical studs (spaced at 16-in on center) are used to attach all gypsum walls to the framing. The gypsum wallboard panels are assumed to be positioned vertically. Note also that let-in diagonal braces are also included at various locations for construction purposes (see Appendix A). These let-in braces were not included in the model as their lateral stiffness was insignificant.

The floor diaphragm of the small house is composed of 2x6 joists spaced at 16 in on center and spanning up to 9 feet along with solid 1 x 6 diagonal sheathing. The floor is supported by 4x6 girders sitting on pier blocks. The roof diaphragm is built with composite shingle felt and solid 1x6 straight sheathing. The ceiling is made of ½ in thick gypsum wallboard.

3.2 Description of Construction Variants

Three construction variants are defined for the small-house index building. The variants are poor-quality, typical-quality, and superior-quality instance of this building. The characteristics of each construction variant are described in sections 3.2.1 to 3.2.3. These characteristics are summarized in Table 3.1 (Porter, 2001).

Table 3.1 Summaries of Construction Variants for Small-House Index Building.

Superior Quality	Typical Quality	Poor Quality
Concrete Stem Wall	Unbraced Cripple Wall with Average-Quality Stucco 80% of stiffness and strength from high-quality laboratory tests	Unbraced Cripple Wall with Poor-Quality Stucco 55% of stiffness and strength from high-quality laboratory tests
Good Quality Stucco 100% of stiffness and strength from high-quality laboratory tests	Average Quality Stucco 90% of stiffness and strength from high-quality laboratory tests	Poor-Quality Stucco 70% of stiffness and strength from high-quality laboratory tests
Superior Nailing of interior gypsum wallboard 100% of stiffness and strength from high-quality laboratory tests	Good Nailing of interior gypsum wallboard 85% of stiffness and strength from high-quality laboratory tests	Poor Nailing of interior gypsum wallboard 75% of stiffness and strength from high-quality laboratory tests

3.2.1 Characteristics of Poor-Quality Variant

1. **Unbraced cripple wall with poor-quality stucco**. Detail 3/S2 (not 4/S2) from the drawings applies (see Appendix A). There are no let-in braces at the cripple wall level. Anchor bolts are nominally at 6' centers, but all the holes are oversized, many of them have inadequate edge distance, and much of the sill plate has been damaged by fungus or termites or by installation of the sill plate in short pieces. The woven wire of the stucco is not furred out from the building paper. The stucco thickness varies between 1/2" and 7/8" with an average thickness of 5/8". The woven wire is fixed to each stud by nails, but many of the nails (perhaps ¼ of them) are missing or poorly installed. The woven wire is not nailed to the sill plate or the top plate. The strength of the stucco material itself is low, and there is extensive (pre-earthquake) deterioration of the stucco. To model these characteristics, it is assumed that the strength of the unbraced cripple walls is 50% to 60% that which would be observed from laboratory tests of a high-quality facsimile.
2. **Poor quality stucco above the first floor**. As with the stucco at the cripple-wall level, the woven wire is not furred out from the building paper, the stucco thickness is less than that shown on the drawings, and many of the nails connecting the woven wire to the studs are missing or poorly installed. To model this aspect of the poor-quality variant, it is assumed that the strength of the exterior stucco wall is 65% to 75% that which would be observed in high-quality laboratory tests.
3. **Poor nailing of interior gypsum wallboard**. Many missing nails to studs, sill, and top plates. The gypsum wallboard is nailed to the studs with a mixture of drywall nails and common nails, with many nails (particularly the common nails) over-driven. The result for modeling purposes is that the strength of the interior wallboard partitions is 75% of what would be observed in a laboratory test of similar thickness wallboard nailed with drywall nails at 7-in centers.
4. **Missing, cut, or split let-in braces.** It appears to be unlikely that let-in braces would be effective in this house regardless of their quality of construction or post-construction deterioration. If the braces were to contribute significantly to the building stiffness, it could be assumed that one in four braces is missing or otherwise inactive.

3.2.2 Characteristics of Typical-Quality Variant

1. **Unbraced cripple wall with average-quality stucco**. Detail 3/S2 (not 4/S2) from the drawings applies (see Appendix A). There are no let-in braces at the cripple wall level. Anchor bolts nominally at 4' centers, but some of holes are oversized, some have inadequate edge distance, and some of the sill plate has been damaged by fungus or termites or by installation of the sill plate in short pieces. The woven wire is adequately furred out from the building paper. The stucco thickness is 7/8". The woven wire is fixed to each stud by nails, but some of the nails are missing or poorly installed. The woven wire is nailed to the sill plate and the top plate. There is some pre-existing deterioration of the stucco. To model these characteristics, it is assumed that the strength of the cripple walls is about 80% of what a laboratory test would suggest.
2. **Average-quality stucco above the first floor**. As with the stucco at the cripple-wall level, the woven wire is furred out from the building paper, the stucco thickness matches that shown on the drawings, and some of the nails connecting the woven wire to the studs are missing or poorly installed. To model this aspect of the typical-quality variant, it is assumed that the strength of the first-floor stucco wall is 90% that which would be observed from laboratory tests.
3. **Good nailing of interior gypsum wallboard**. Local missing nails to studs, sill, and top plates. Few nails are over-driven. For modeling purposes, it is assumed that the strength of the wall is 80% to 90% of laboratory test results for similar construction.
4. **Good let-in braces.** It appears that the let-in braces would not contribute significantly to stiffness or strength, but if they did, it could be assumed that one in 10 let-in braces are inactive because of splitting.

3.2.3 Characteristics of Superior-Quality Variant

1. **Concrete Stem wall.** A concrete foundation stem wall replaces the wood framed cripple in this variant. Detail 4/S2 applies, not 3/S2 (see Appendix A). Anchor bolts are installed correctly and are in good condition. The sill is in good condition. The rim joist is properly nailed to the sill from the outside, rather than from the inside.
2. **Good quality stucco**. The woven wire is adequately furred out from the building paper. The stucco thickness is 7/8". The woven wire is fixed to each stud by nails as shown in the drawings. The woven wire is nailed to the sill plate and the top plate at 6" centers. No pre-existing deterioration exists. Strength approximately matches that exhibited by high-quality laboratory tests.
3. **Good nailing of interior gypsum wallboard**. No missing or over-driven nails to studs, sill, or top plates. For modeling purposes, the strength of the gypsum wallboard partitions matches that of high-quality laboratory tests.
4. **Good let-in braces.** All let-in braces are effective.

3.3 Modeling Assumptions

The main assumptions used to develop the numerical pancake model of each construction variant of the small-house index building are briefly discussed in this section.

Each stucco wall, gypsum wall, cripple and/or concrete stem wall is modeled by an in-plane shear element exhibiting the Wayne Stewart hysteresis rule, as described in Chapter 2. All exterior walls are sheathed with stucco on the outside and gypsum on the inside. Two parallel shear elements are used to simulate the in-plane behavior of both sheathing materials. All interior walls are sheathed with gypsum on both sides and only one shear element per wall line is considered. Preliminary calculations showed that the lateral stiffness provided by the let-in braces is much smaller than that of the gypsum and stucco walls. Therefore, the let-in braces are not considered in the model.

The in-plane behavior of the floor diaphragm is modeled by linear-elastic quadrilateral finite elements. The in-plane stiffness of the roof diaphragm is neglected as it was deemed that straight sheathing boards are not able to provide substantial in-plane stiffness. Therefore, only the in-plane stiffness of the gypsum ceiling is modeled by linear-elastic quadrilateral finite elements.

3.4 Node Numbering

The pancake model of the small-house index building incorporates three layers of nodes, as illustrated in Figure 3.2.

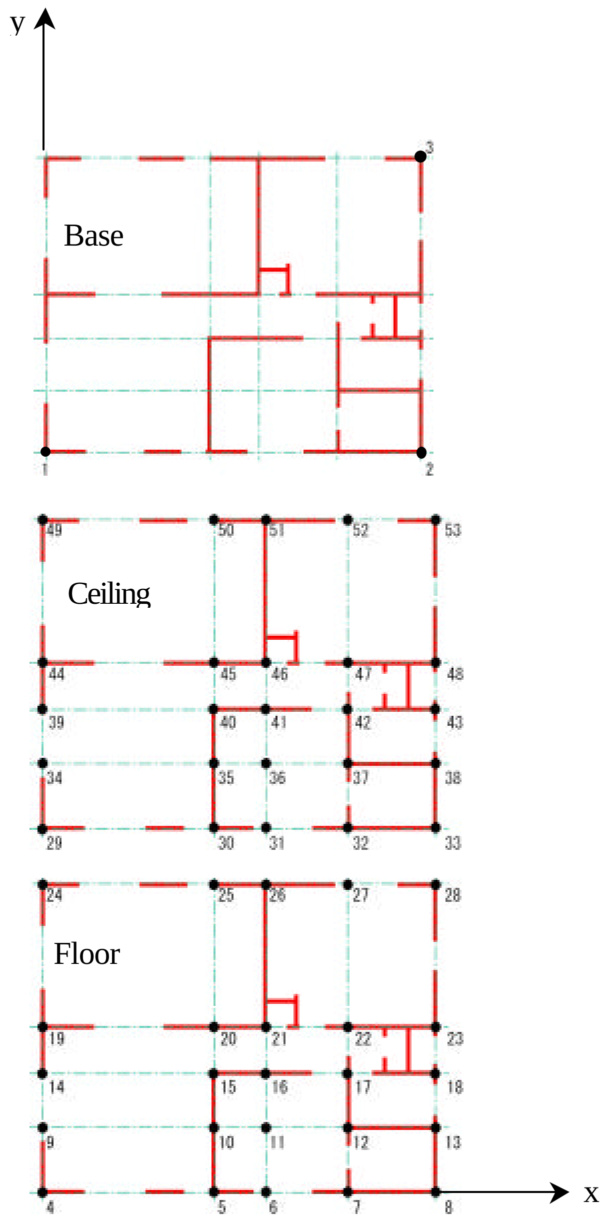


Figure 3.2 Node Numbering for Pancake Model of Small-House Index Building.

The three base nodes are located below the foundation wall (cripple or stem) of the building. The earthquake ground motion is applied simultaneously at these three nodes. The floor nodes are located at the level of the floor diaphragm and are used to connect the shear elements representing the foundation walls from the base level to the floor diaphragm and to connect also the interior and exterior shear elements representing the vertical walls between the floor diaphragm and the ceiling level. The ceiling nodes are used to connect the interior and exterior shear elements representing the vertical walls between the floor diaphragm and the ceiling level.

3.5 Elements Description and Location

The various elements used to represent the lateral load resisting system of the small-house index building are briefly described in this section.

Four shear elements are used to represent the exterior foundation walls (stem or cripple) of the small-house index building. Figure 3.3 shows the location, orientation and number assigned to each of these shear elements in the RUAUMOKO data files.

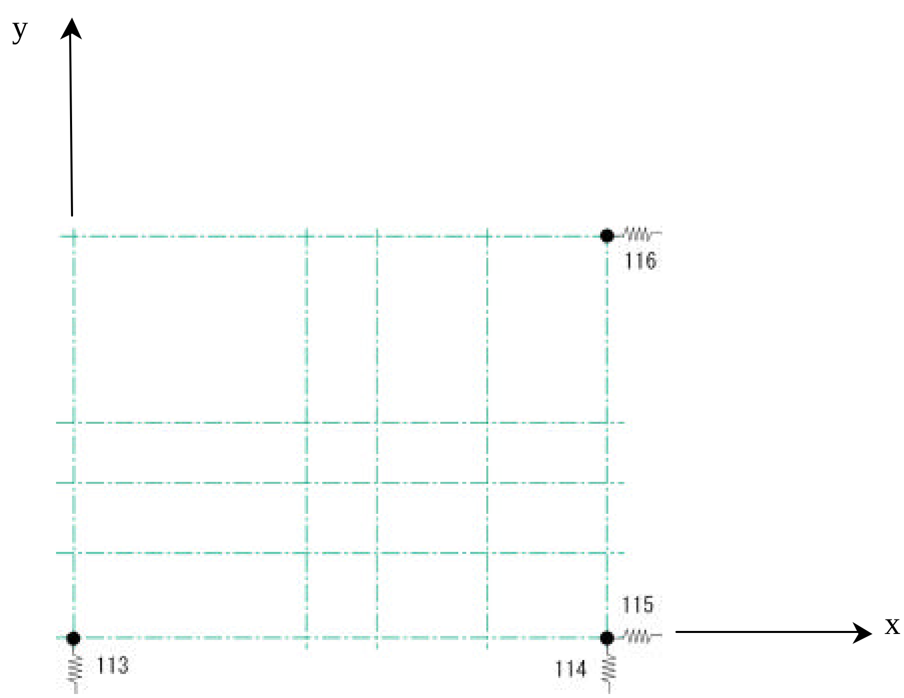


Figure 3.3 Location, Orientation and Numbering of Shear Elements for Foundation Walls (stem or cripple) of the Small-House Index Building.

The location, orientation and number assigned to the 14 shear elements used to represent the interior and exterior vertical walls of the small-house index building are illustrated in Figure 3.4. To simplify the model, the interior wall lines 1.3, 1.6, and 2.1 have been lumped with wall lines 1, 2, and 3, respectively.

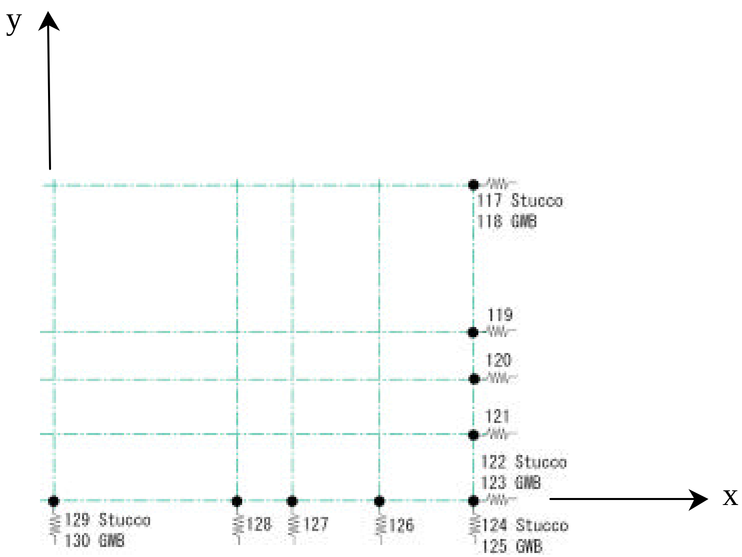


Figure 3.4 Locations, Orientation and Numbering of Shear Elements for Interior and Exterior Vertical Walls of the Small-House Index Building.

The location, orientation and number assigned to the 32 linear-elastic quadrilateral finite elements used to represent the floor and ceiling diaphragms of the small-house index building are illustrated in Figure 3.5. The first number for each element represents the floor diaphragm element, while the second number represents the roof diaphragm element.

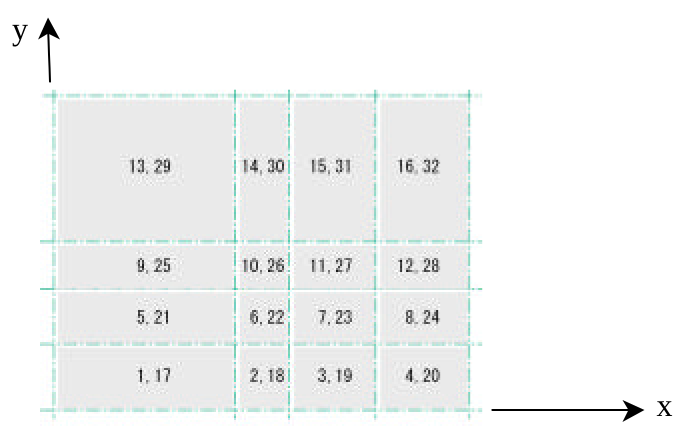


Figure 3.5 Locations, Orientation and Numbering of the finite elements representing the floor and ceiling diaphragms of the Small-House Index Building.

The location, orientation and number assigned to the linear-elastic frame elements used along the edges of the floor and ceiling diaphragms to connect the corners of the quadrilateral finite elements to the vertical wall elements are illustrated in Figure 3.6.

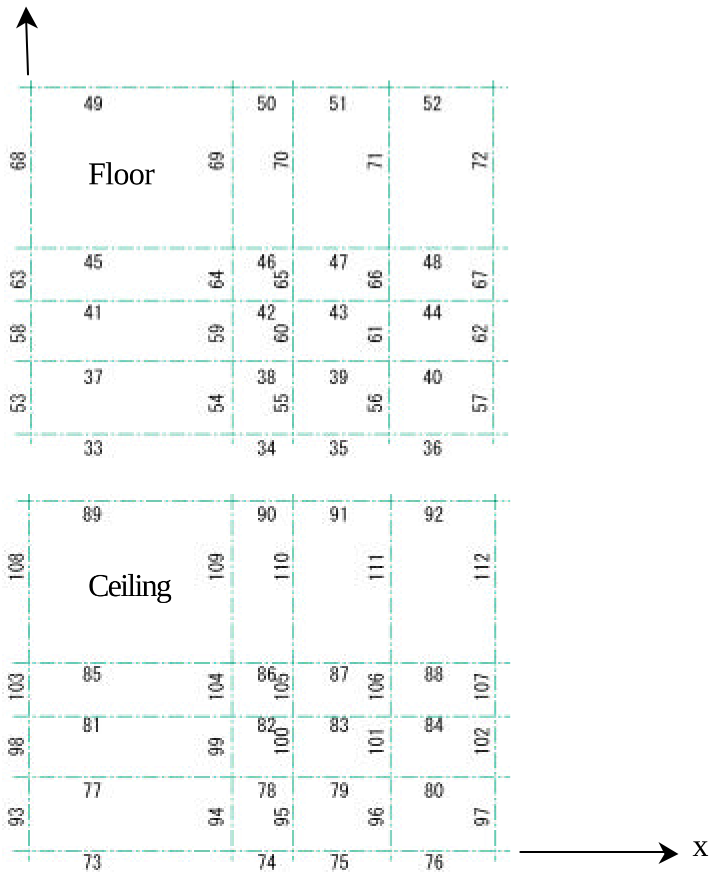


Figure 3.6 Locations, Orientation and Numbering of Frame Elements Along the Edges of the Floor and Ceiling Diaphragms of the Small-House Index Building.

3.6 Hysteretic Parameters for Foundation and Vertical Wall Shear Elements

The in-plane cyclic response of the foundation and vertical walls incorporated in the small-house index building were modeled by shear elements exhibiting the Wayne Stewart Hysteresis rule. As shown in Figure 2.2, this hysteresis law is defined by nine independent physical parameters: F_y = Equivalent lateral yield strength; k_o = Initial lateral stiffness; R_f = Post-yield stiffness factor; F_u = Ultimate lateral capacity; F_i = Intercept force; PTRI = Tri-linear stiffness factor beyond the ultimate capacity; PUNL = Unloading stiffness factor; b_1 = Softening factor; and

a_1 = Reloading stiffness factor. These parameters for each cripple, stucco, and gypsum wall were estimated from available cyclic test data on wall assemblies.

Figure 3.7 shows hysteresis loops obtained from one of three cyclic tests on 8 ft x 8 ft walls sheathed with 7/8-in thick stucco on one side. These tests were conducted at the University of California at Irvine for the City of Los Angeles (COLA) Project (Pardoen, 2000). The wire mesh was attached to the wood framing by furring nails (3/8-in head) spaced at 6 in on center along the vertical studs (spaced at 16 in on center) and at 6 in along the top and bottom plate. Table 3.2 shows the resulting parameters of the Wayne Stewart hysteresis rule extracted from these test results. The hysteretic properties assigned to the shear elements representing the exterior vertical stucco walls (Elements 117, 122, 124 and 129 in Fig. 3.4) were obtained by adjusting the strength and stiffness values in Table 3.2 for the actual length of full wall piers in each wall line. The resulting properties are shown in Tables 3.3, 3.4, and 3.5 for the poor-quality, typical-quality and superior-quality variant, respectively.

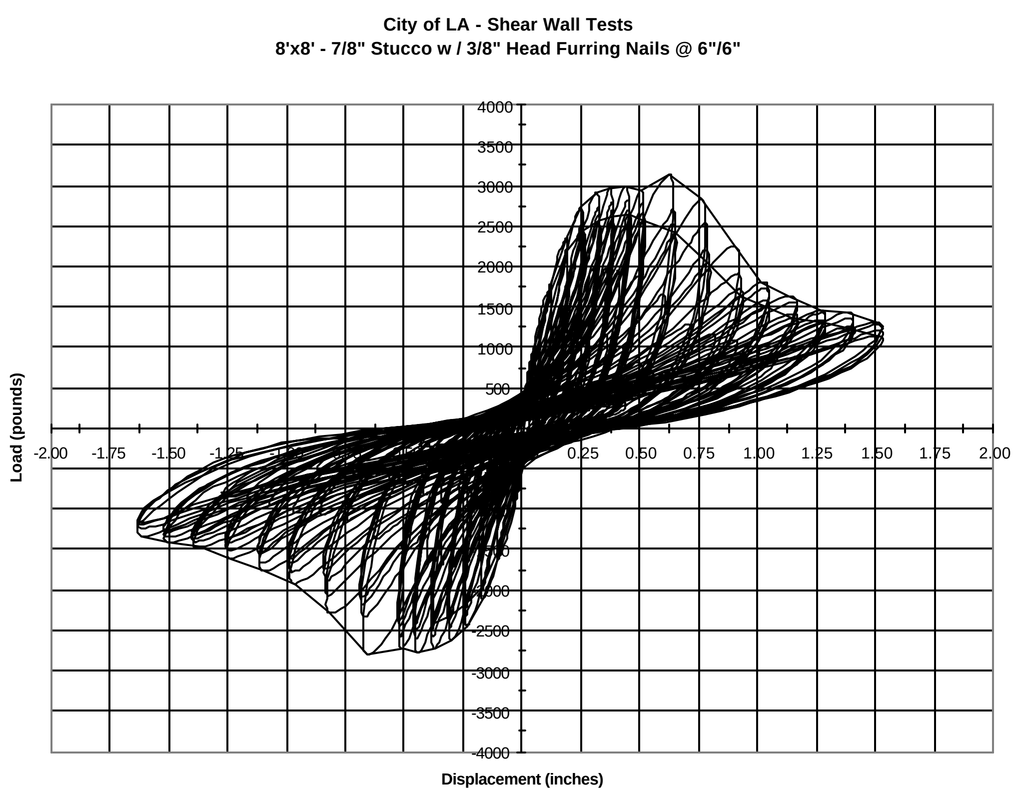


Figure 3.7 Hysteresis Loops From Cyclic Test on 8 ft x 8 ft Wall Sheathed with 7/8-in Thick Stucco on One Side (from Pardoen, 2000).

Figure 3.8 shows hysteresis loops obtained from one of three cyclic tests on 8 ft x 8 ft walls sheathed with ½-thick gypsum wallboard on both sides. These tests were conducted at the University of California at Irvine for the City of Los Angeles (COLA) Project (Pardoen, 2000). The gypsum wallboard was attached to the wood framing by 1-5/8 in drywall nails spaced at 7 in on center along the vertical studs (spaced at 16 in on center) and at 7 in along the top and bottom plate. Table 3.2 shows the resulting parameters of the Wayne Stewart hysteresis rule extracted from these test results. The hysteretic properties assigned to the shear elements representing the interior vertical gypsum walls (see Figure 3.3) were obtained by adjusting the strength and stiffness values in Table 3.2 for the actual length of full wall piers in each wall line. The same procedure was adopted for the interior gypsum sheathing of the exterior walls but the values were divided by two since these walls are sheathed with gypsum wallboard only one side. The resulting properties are shown in Tables 3.3, 3.4, and 3.5 for the poor-quality, typical-quality and superior-quality variant, respectively.

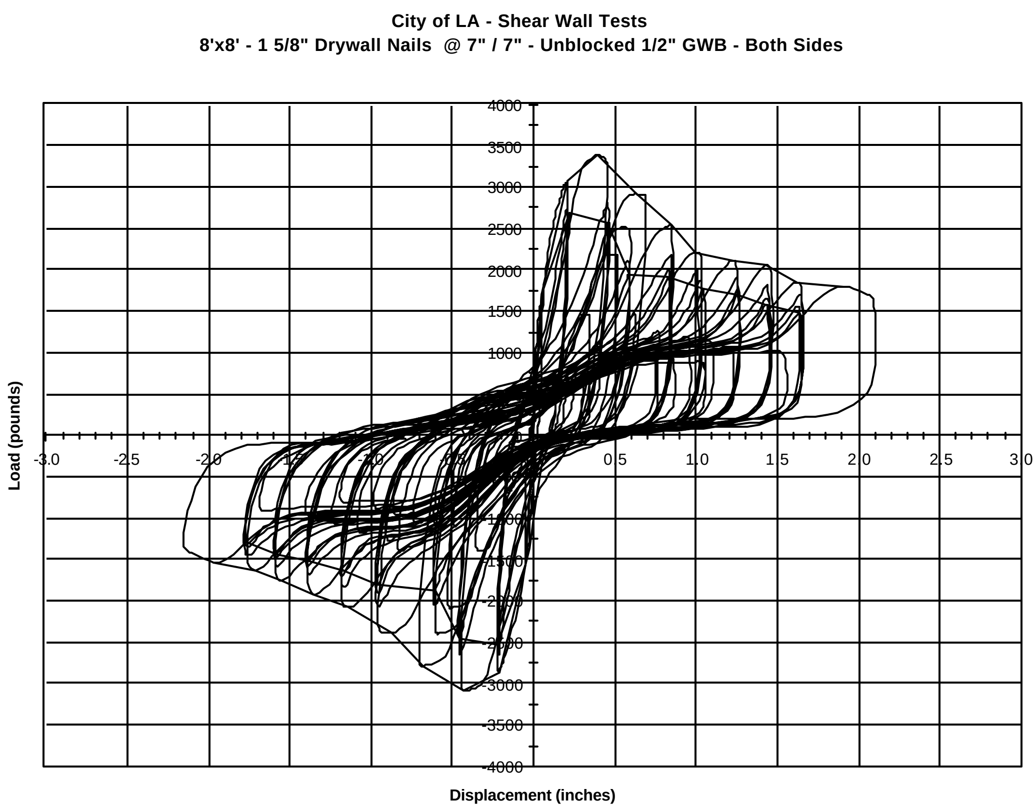


Figure 3.8 Hysteresis Loops From Cyclic Test on 8 ft x 8 ft Wall Sheathed with 1/2-in Thick Gypsum Wallboards on Both Sides (from Pardoen, 2000).

Figure 3.9 shows hysteresis loops obtained from two cyclic tests on 2 ft high x 12 ft long leveled cripple walls. These tests were conducted at the University of California at Davis under Task 1.4.3 of the CUREE-Caltech Woodframe Project (Chai, 2000). The first test specimen was sheathed with 15/32" in thick OSB on one side and 7/8-in thick stucco on the other side. The second test specimen was sheathed with OSB only. In order to isolate the contribution of the stucco alone, the load values of these two hysteresis loops were subtracted to obtain an estimate of the hysteretic response of a cripple wall sheathed with stucco only. Table 3.2 shows the resulting parameters of the Wayne Stewart hysteresis rule extracted from these test results. The hysteretic properties assigned to the shear elements representing the exterior cripple walls (see Figure 3.2) were obtained by adjusting the strength and stiffness values of Table 3.2 for the actual length of cripple walls in each wall line.

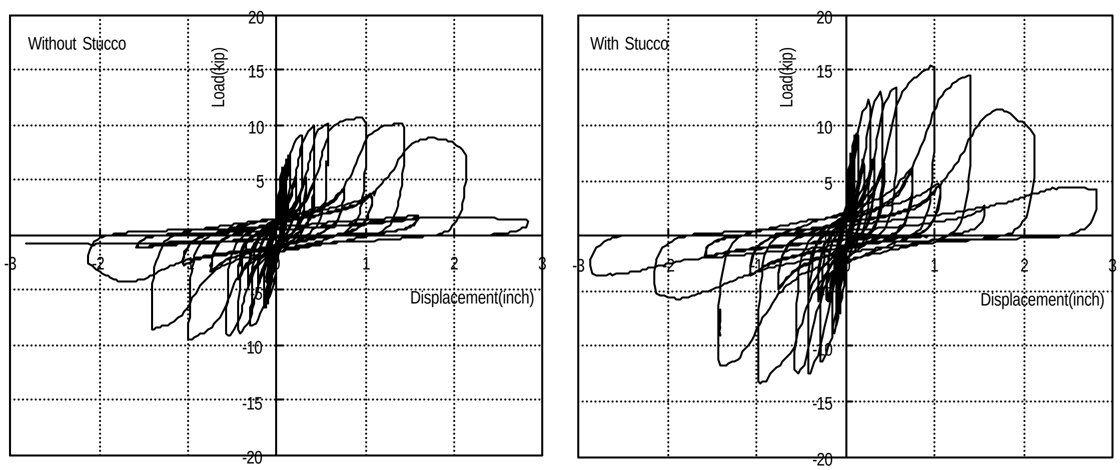


Figure 3.9 Hysteresis Loops From Two Cyclic Tests on 2 ft High x 12 ft Long Cripple Walls (from Chai, 2000).

The shear elements representing the concrete stem walls of the superior-quality variant are assigned linear-elastic properties based on the shear stiffness of the wall line:

$$k_s = \frac{G_c A}{l}$$

where k_s is the in-plane lateral stiffness of the stem wall, G_c is the shear modulus of concrete, A is the cross-sectional area of the stem wall, and l is the total length of the stem wall line.

The resulting properties are shown in Tables 3.3, 3.4, and 3.5 for the poor-quality, typical-quality and superior-quality variant, respectively.

Table 3.2 Parameters of Wayne Stewart Hysteresis Rule Obtained from Experimental Results (Figs. 3.6 to 3.8).

Wall Type	Length (in)	Sheathing Sides	F_y (kips)	k_o (kips/in)	R_f	F_u (kips)	F_i (kips)	PTRI
Cripple Stucco	145.5	1	2.87	38.0	0.018	4.30	0.72	-0.037
Vertical Stucco	96	1	1.94	22.3	0.082	2.91	0.49	-0.064
Vertical Gypsum	96	2	2.31	38.0	0.034	3.46	0.58	-0.047
PUNL = 1.45, $a_1 = 0.38$ and $b_1 = 1.09$ for all wall types								

Table 3.3 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Poor-Quality Variant of the Small-House Index Building.

Wall Type	Wall Line	Shear Element No.	F_y (kips)	k_o (kips/in)	R_f	F_u (kips)	F_i (kips)	PTRI
Cripple Stucco	A, C	115, 116	5.20	69.0	0.018	7.80	1.30	-0.037
	1,5	113, 114	3.90	51.7	0.018	5.85	0.98	-0.037
Vertical Exterior Stucco	A	117	4.20	48.3	0.082	6.30	1.05	-0.064
	C	122	4.24	48.8	0.082	6.36	1.06	-0.064
	1	124	3.61	41.5	0.082	5.42	0.90	-0.064
	5	129	3.01	34.6	0.082	4.52	0.75	-0.064
Vertical Gypsum	A	118	2.29	44.1	0.034	3.44	0.57	-0.047
	B	119	4.92	94.6	0.034	7.38	1.23	-0.047
	B1	120	2.94	56.5	0.034	4.41	0.74	-0.047
	B2	121	1.48	28.5	0.034	2.22	0.37	-0.047
	C	123	2.32	44.5	0.034	3.47	0.58	-0.047
	1	125	2.90	55.7	0.034	4.35	0.73	-0.047
	2	126	2.39	45.9	0.034	3.58	0.60	-0.047
	3	127	3.29	63.3	0.034	4.94	0.82	-0.047
	4	128	2.22	42.8	0.034	3.34	0.56	-0.047
	5	130	1.65	31.6	0.034	2.47	0.41	-0.047
PUNL = 1.45, $a_1 = 0.38$ and $b_1 = 1.09$ for all shear elements								

Table 3.4 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Typical-Quality Variant of the Small-House Index Building.

Wall Type	Wall Line	Shear Element No.	F_y (kips)	k_o (kips/in)	R_f	F_u (kips)	F_i (kips)	PTRI
Cripple Stucco	A, C	115, 116	7.57	100	0.018	11.4	1.89	-0.037
	1,5	113, 114	5.67	75.2	0.018	8.51	1.42	-0.037
Vertical Exterior Stucco	A	117	5.40	62.1	0.082	8.10	1.35	-0.064
	C	122	5.45	62.7	0.082	8.17	1.36	-0.064
	1	124	4.64	53.4	0.082	6.97	1.16	-0.064
	5	129	3.87	44.5	0.082	5.81	0.97	-0.064
Vertical Gypsum	A	118	2.60	50.0	0.034	3.90	0.65	-0.047
	B	119	5.58	107	0.034	8.37	1.39	-0.047
	B1	120	3.33	64.1	0.034	5.00	0.83	-0.047
	B2	121	1.68	32.3	0.034	2.52	0.42	-0.047
	C	123	2.62	50.4	0.034	3.94	0.66	-0.047
	1	125	3.29	63.2	0.034	4.93	0.82	-0.047
	2	126	2.71	52.0	0.034	4.06	0.68	-0.047
	3	127	3.73	71.8	0.034	5.60	0.93	-0.047
	4	128	2.52	48.5	0.034	3.78	0.63	-0.047
	5	130	1.87	35.9	0.034	2.80	0.47	-0.047
PUNL = 1.45, $a_1 = 0.38$ and $b_1 = 1.09$ for all shear elements								

Table 3.5 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Superior-Quality Variant of the Small-House Index Building.

Wall Type	Wall Line	Shear Element No.	F_y (kips)	k_o (kips/in)	R_f	F_u (kips)	F_i (kips)	PTRI
Stem Concrete	A, C	115, 116	Linear-Elastic	272,000	0.018	Linear-Elastic	Linear-Elastic	-0.037
	1,5	113, 114		204,000	0.018			-0.037
Vertical Exterior Stucco	A	117	6.00	69.0	0.082	9.00	1.50	-0.064
	C	122	6.06	69.6	0.082	9.08	1.51	-0.064
	1	124	5.16	59.4	0.082	7.74	1.29	-0.064
	5	129	4.30	49.5	0.082	6.45	1.08	-0.064
Vertical Gypsum	A	118	3.06	58.8	0.034	4.59	0.76	-0.047
	B	119	6.56	126	0.034	9.85	1.64	-0.047
	B1	120	3.92	75.4	0.034	5.88	0.98	-0.047
	B2	121	1.98	38.0	0.034	2.97	0.49	-0.047
	C	123	3.09	59.3	0.034	4.63	0.77	-0.047
	1	125	3.87	74.3	0.034	5.80	0.97	-0.047
	2	126	3.18	61.2	0.034	4.78	0.80	-0.047
	3	127	4.39	84.4	0.034	6.59	1.10	-0.047
	4	128	2.97	57.0	0.034	4.45	0.74	-0.047
	5	130	2.19	42.2	0.034	3.29	0.55	-0.047
PUNL = 1.45, $a_1 = 0.38$ and $b_1 = 1.09$ for all shear elements								

3.7 Properties of Horizontal Floor and Roof Diaphragms

The in-plane stiffness of the floor diaphragm, G_d, is taken to be 500 kips/in, while the corresponding value for the ceiling diaphragm is take to be 8 kips/in. These values are prescribed by the NEHRP Guidelines for Seismic Rehabilitation of Buildings (FEMA, 1997). Each linear-elastic diaphragm finite element is assigned elastic properties such that:

$$Gt = \frac{Et}{2(1+\boldsymbol{n})} = G_d$$

where G is the equivalent shear modulus, E is the equivalent elastic modulus, $\boldsymbol{n}$ is the equivalent Poisson's ratio and t is the thickness of the finite element.

3.8 Weight Distribution

Table 3.6 lists the weights considered for the small-house index building. These weights were distributed as nodal lumped seismic weights according to the tributary areas of the nodes (see Figure 3.2).

3.9 Description of RUAUMOKO Data Files

The three RUAMOKO data files corresponding to the poor-quality, typical-quality and superior-quality variants are contained in the CD-ROM accompanying this report. These data files are self-contained and include, as ground motion input, one component of the acceleration time-history recorded at Canoga Park during the 1994 Northridge Earthquake. This ground motion is oriented parallel to the short side of the building (y-axis direction).

3.10 Analysis Examples

In this section, the three RUAUMOKO data files are used to evaluate the seismic response of the three variants of the small-house building index when excited along its short side by the Canoga Park record of the 1994 Northridge Earthquake scale to a Peak Ground Acceleration (PGA) of 0.30 g. Figure 3.10 shows the unscaled (PGA = 0.42 g) acceleration time-history of this record.

Table 3.6 Weights for Small-House Index Building.

Item	Location		Length (ft)	Width or Height (ft)	Area (sq ft)	Unit Weight (psf)	Weight #	Total Weight #
Roof			32	42	1344	11.3	15,187	15,187
Ceiling			30	40	1200	5.5	6,600	6,600
Wall	Line A-Rear	Total	40	8	320			3,808
		window/door			62	4	248	
		typ exterior			258	13.8	3,560	
	Line B-Int	Total	40	8	320			1,768
		open & door			80	2	160	
		typ interior			240	6.7	1,608	
	Line B.1-Int	Total	19	8	152			940
		open & door			16.7	2	33	
		typ interior			135.3	6.7	907	
	Line B.2-Int	Total	8	8	64			429
		open & door			0	2	0	
		typ interior			64	6.7	429	
	Line C-Front	Total	40	8	320			3,701
		window/door			73	4	292	
		typ exterior			247	13.8	3,409	
	Line 1-Right	Total	30	8	240			2,900
		window/door			42	4	168	
		typ ext			198	13.8	2,732	
	Line 1.3-Int	Total	5	8	40			268
		open & door			0	2	0	
		typ interior			40	6.7	268	
	Line 1.6-Int	Total	5	8	40			190
		open & door			16.7	2	33	
		typ interior			23.3	6.7	156	
	Line 2-Int	Total	18	8	144			808
		open & door			33.4	2	67	
		typ interior			110.6	6.7	741	
	Line 2.1-Int	Total	18	8	144			604
		closet door			63.3	1	63	
		typ interior			80.7	6.7	541	
	Line 3-Int	Total	14	8	112			750
		open & door			0	2	0	
		typ interior			112	6.7	750	
	Line 4-Int	Total	12	8	96			643
		open & door			0	2	0	
		typ interior			96	6.7	643	
	Line 5-Left	Total	30	8	240			2,900
		window/door			42	4	168	
		typ ext			198	13.8	2,732	
Floor			30	40	1200	5.9	7080	7080
Cripple Walls	Line A	ext	40	2	80	9.9	792	792
	Line C	ext	40	2	80	9.9	792	792
	Line 1	ext	30	2	60	9.9	594	594
	Line 5	ext	30	2	60	9.9	594	594
Total Building Weight								**51,349**

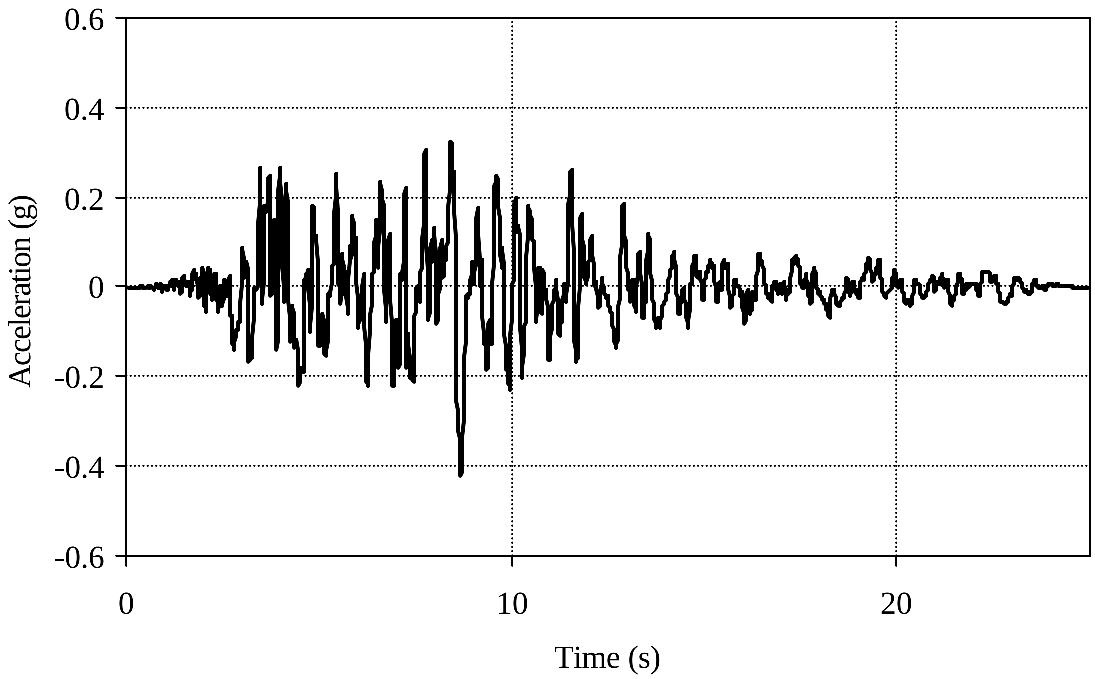


Figure 3.10 Canoga Park Record, 1994 Northridge Earthquake.

Table 3.7 shows the fundamental frequency computed based on the initial stiffness for each of the construction variants. The fundamental frequencies of the poor-quality and typical-quality variants are close to each other due to the presence of cripple walls. The fundamental frequency of the superior-quality variant is much higher due to the significantly greater rigidity of the concrete stem wall.

Table 3.7 Fundamental Frequencies of Small-Index Building.

Construction Variant	Fundamental Frequency (Hz)	Mode of Vibration
Poor-Quality	4.35	Y-direction
Typical-Quality	5.13	Y-direction
Superior-Quality	10.5	Y-direction

Figure 3.11 shows, for the three construction variants, the displacement time-histories in the y-direction at the floor and ceiling level, respectively. The detrimental effect of the cripple walls in the poor-quality and typical-quality variants can be clearly seen from the figures. For these two construction variants, practically all the displacements are the results of the cripple wall deformations. The introduction of the concrete stem wall in the superior-quality variant reduces the displacements at the floor and ceiling levels by an order of magnitude.

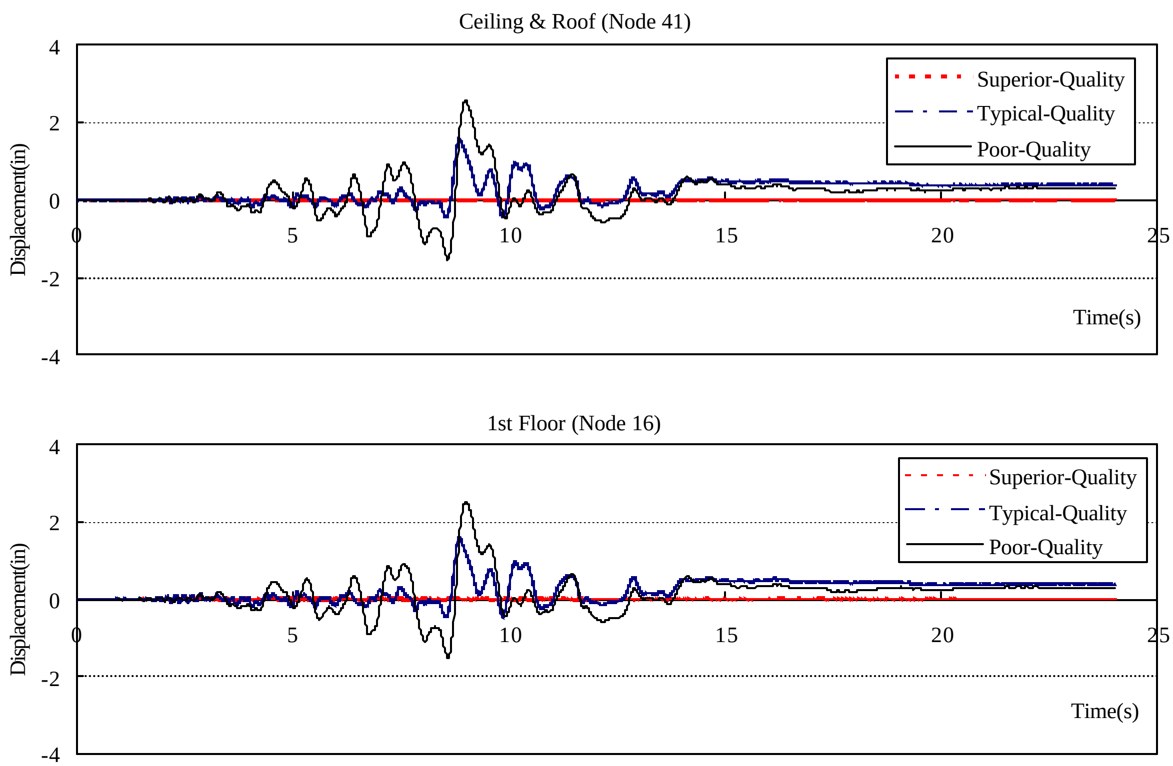


Figure 3.11 Displacement Time-Histories in the Y-Direction for Small-House Index Building Under Canoga Park Record, PGA = 0.30 g.

Figure 3.12 presents, for the three construction variants, the hysteresis loops of the foundation wall located along wall line 5 of the index building (element 113). Large inelastic deformations occur in the poor-quality and typical–quality variants. For the poor-quality variant, the maximum displacement exceeds the displacement at ultimate load of the cripple wall. The much higher lateral stiffness of the concrete stem wall is also evident.

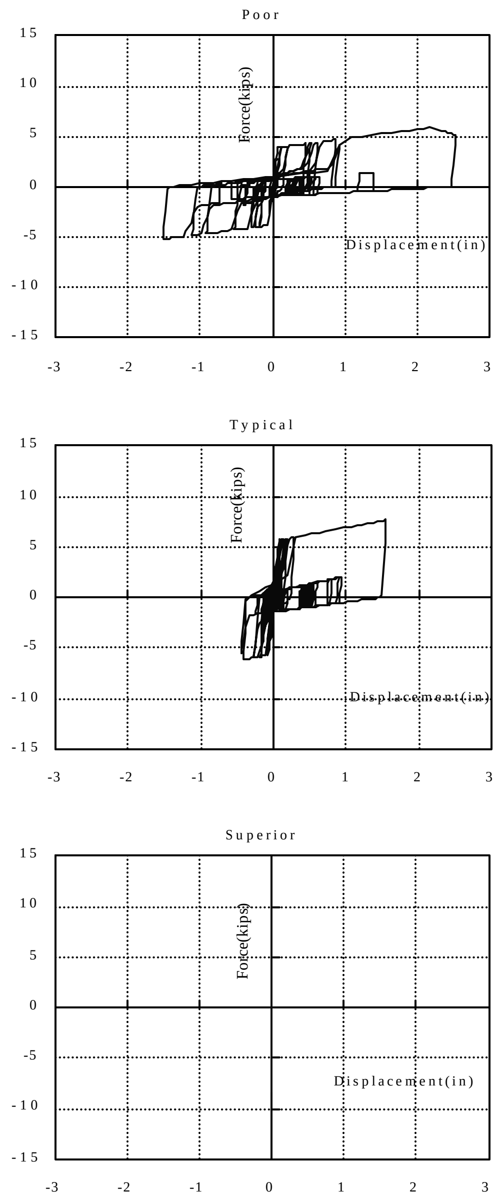


Figure 3.12 Hysteresis Loops of Foundation Walls Along Wall Line 5 (Element 113) for Small-House Index Building Under Canoga Park Record, PGA = 0.30 g.

3.11 Retrofit of Small House: Retrofit Measure No. 1

Only one retrofit measure was considered for the small house index building. The cripple walls for the poor-quality and typical-quality variants were reinforced with OSB sheathing along their length as shown in Figure 3.13 (Russell 2001). The two RUAMOKO data files incorporating this cripple wall retrofit for the poor-quality and typical-quality variants are included in the CD-ROM accompanying this report. These data files are self-contained and include, as ground motion input, one component of the acceleration time-history recorded at Canoga Park during the 1994 Northridge Earthquake. This ground motion is oriented along the short side of the building.

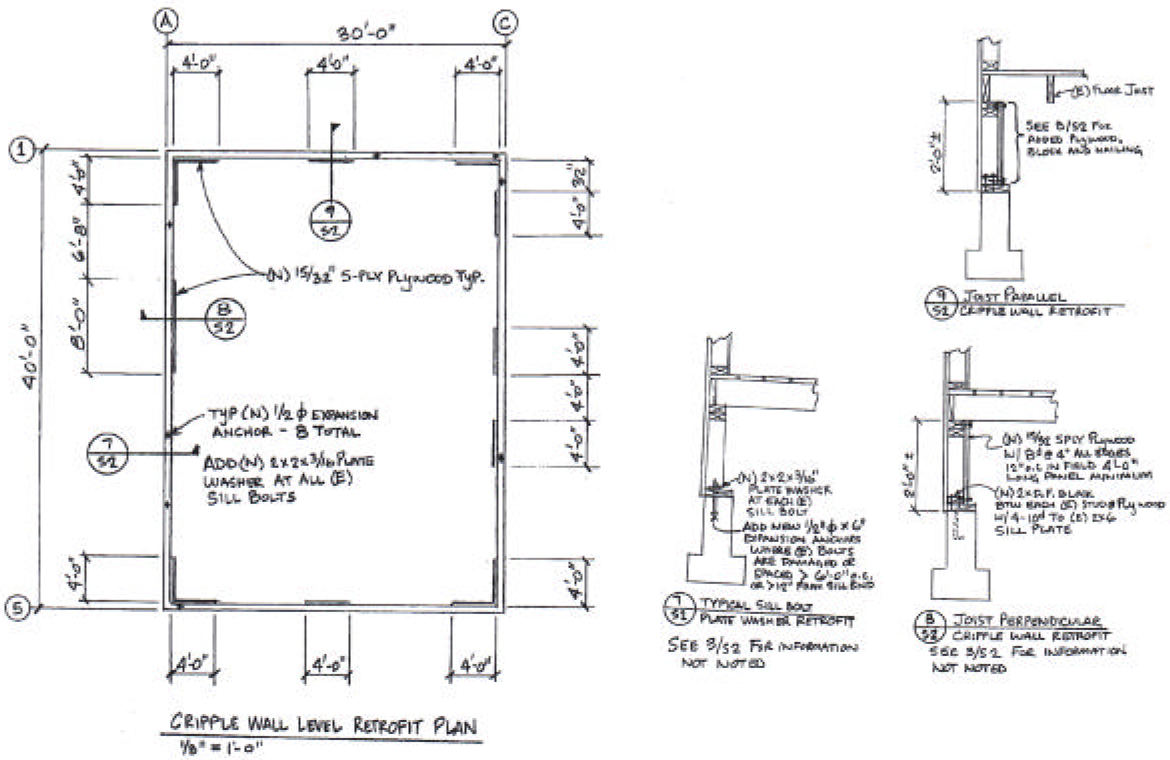


Figure 3.13 Retrofit of Cripple Walls for Typical-Quality and Poor-Quality Variants of the Small House Index Building (Russell 2001).

In order to model the retrofitted cripple walls, shear elements 131 to 134 were added, as shown in Figure 3.14.

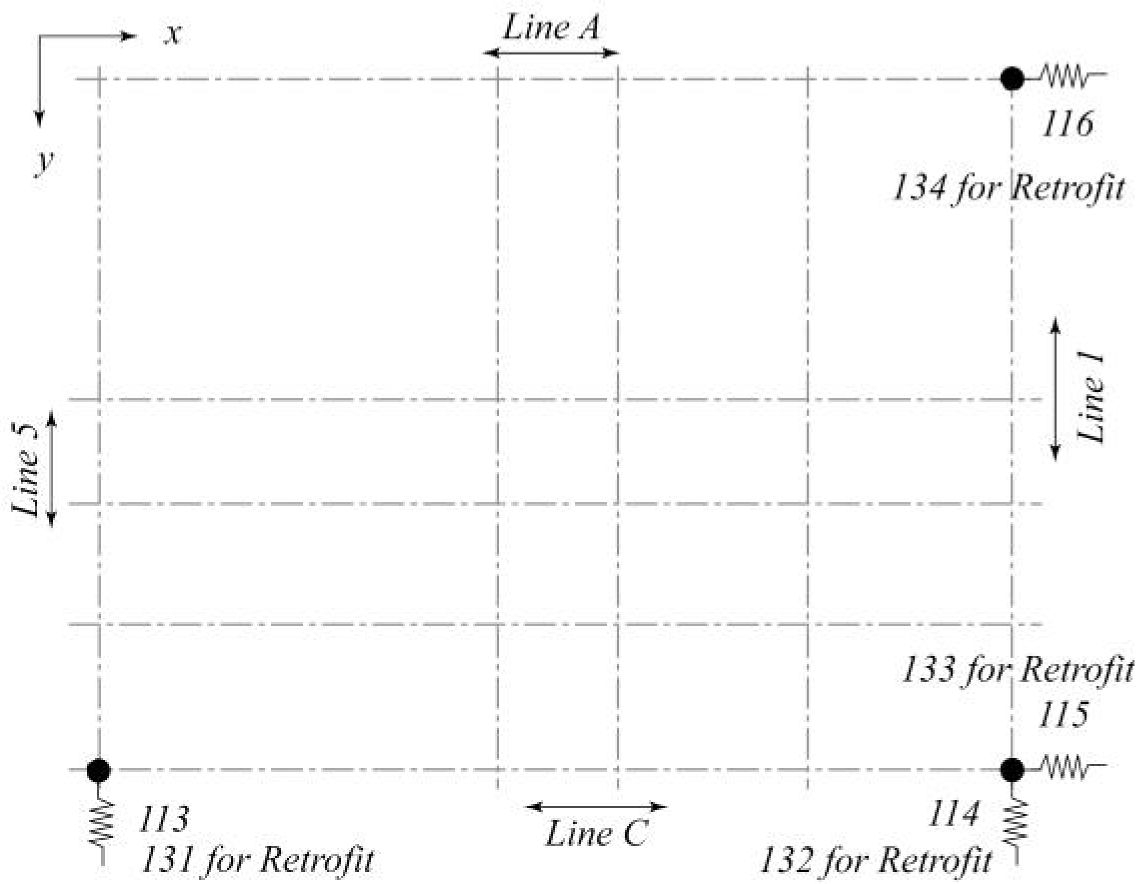


Figure 3.14 Location, Orientation and Numbering of the Shear Elements used to model the Retrofitted Cripple walls for the Small House Index Building.

The computer program CASHEW (Folz and Filiatrault, 2001) was used to determine the cyclic response of the shear element representing each braced cripple wall. Table 3.7 lists parameters of the sheathing-to-framing connectors used in the CASHEW program.

Table 3.7 Sheathing-to Framing Connector Parameters Used to Model Braced Cripple Walls of Retrofitted Small-House Index Building with the CASHEW program.

k_o (kips/in)	r_1	r_2	r_3	r_4	F_0 (kips)	F_i (kips)	D_u (in)	a_1	b
4.87	0.049	-0.049	1.40	0.015	0.18	0.042	0.50	0.80	1.10

Figure 3.15 presents the envelope of the lateral load-displacement relationship for each braced cripple wall line as predicted by the CASHEW program.

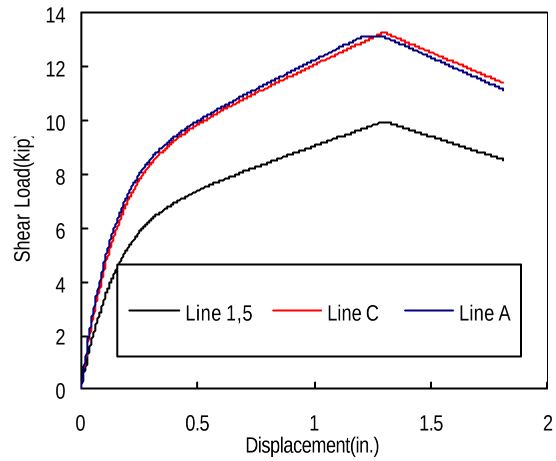


Figure 3.15 Envelope of Lateral Load-Displacement Relationship of OSB Cripple Walls of Retrofitted Small House Index Building as Predicted by the CASHEW Program.

Table 3.8 presents the resulting parameters used to model the OSB cripple walls.

Table 3.8 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements Representing the OSB Cripple Walls of the Small-House Index Building.

	Line A	Line C	Line 1,5
Fy (Kip)	8.17	8.11	6.08
Ko (Kip/in)	65.01	60.63	45.47
RF(r1)	0.063	0.066	0.066
Fu (Kip)	13.12	13.27	9.96
FI (Kip)	1.94	1.96	1.47
PTRI	-0.059	-0.061	-0.061
PUNL	1.22	1.21	1.21
ALPHA	0.78	0.77	0.77
BETA	1.10	1.10	1.10

Table 3.9 shows the fundamental frequencies of the original and retrofitted poor-quality and typical quality variants of the small-house index building. Retrofitting the cripple walls with OSB sheathing increases significantly the fundamental frequency of each construction variant.

Table 3.9. Fundamental Frequencies of Original and Retrofitted Small House Index Building.

Construction Variant	Fundamental Frequency (Hz)	
	Original Small House	Retrofitted Small House
Poor-Quality	4.35	5.58
Typical-Quality	5.13	6.13

The retrofitted poor-quality and typical-quality variants of the small house index building were again excited along their short sides by the Canoga Park record of the 1994 Northridge Earthquake scale to a Peak Ground Acceleration (PGA) of 0.30 g (see Fig. 3.9). Figure 3.16 shows, for the original and retrofitted poor-quality and typical-quality variants, the displacement time-histories in the y-direction at the ceiling level. Retrofitting the cripple walls of both construction variants with OSB sheathing reduces dramatically the displacements.

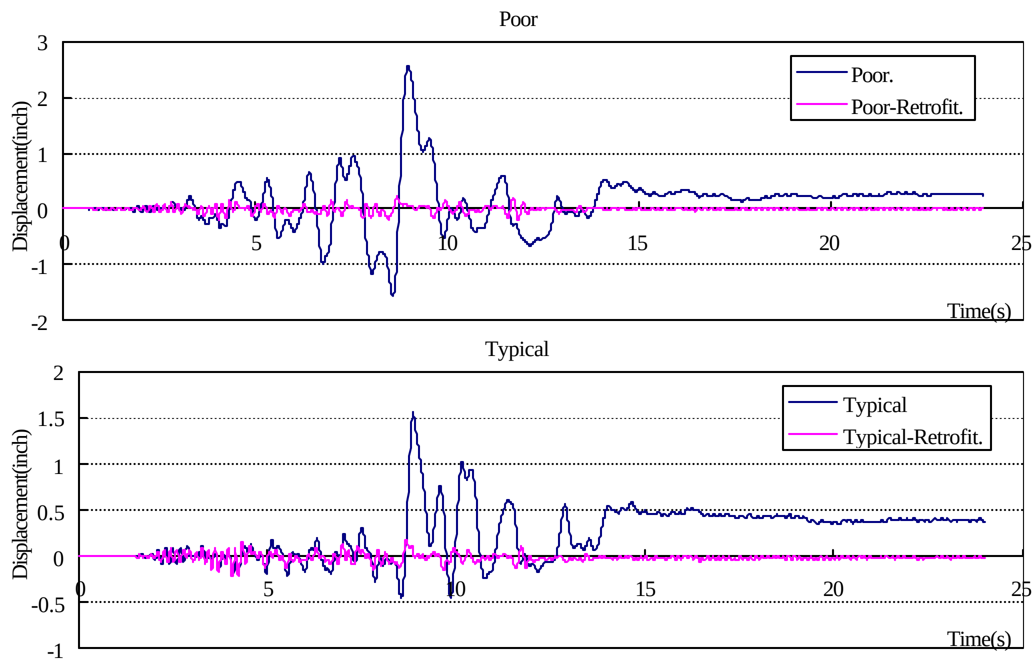


Figure 3.16 Displacement time-histories at ceiling level (Node 41) in the Y-direction for Original and Retrofitted Small House Index Building Under Canoga Park Record, PGA = 0.30 g.

4. MODELING OF INDEX BUILDING 2: LARGE HOUSE

4.1 General Description

The second index building considered represents a two story single family dwelling of approximately 2400 square feet. This building is assumed to have been built as a housing development “production house” in either the 1980’s or 1990’s. Figure 4.1 shows plan views of the building. The architectural and structural drawings of the large-house index building are included in Appendix B (Cobeen, 2001).

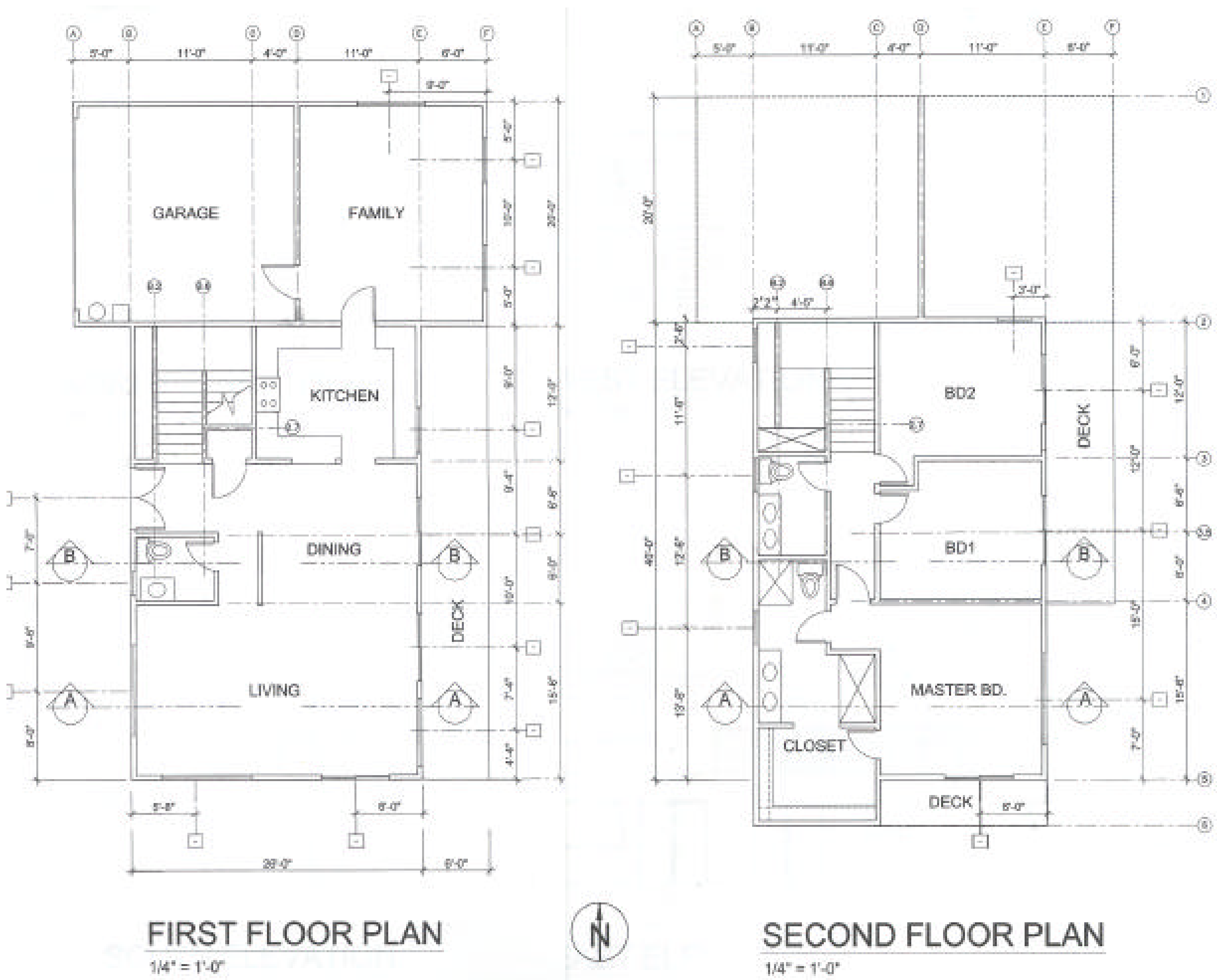


Figure 4.1 Plan Views of Large-House Index Building.

The exterior walls of the large-house index building are sheathed with stucco (3-coat 7/8-in thick) on the outside along with wood shear walls (7/16 in thick OSB) and gypsum wallboards (1/2-in thick) on the inside. One-inch crown staples, spaced at 6 in on center along the vertical studs, are used to attach the wire mesh of the stucco finish to the wood framing. Eight-penny common nails spaced at 6, 4 or 3 in along the edges and 12 in on the field are used to attach the OSB panels to the framing. All interior gypsum walls are sheathed on both sides. Drywall nails (1-5/8 in long) spaced at 7 in on center along the vertical studs (spaced at 16-in on center) are used to attach all gypsum walls to the framing. The gypsum walls are assumed to be positioned vertically. Note also that let-in diagonal braces are also included at various locations for construction purposes (see Appendix B). These let-in braces were not included in the model.

The floor diaphragm of the large-house index building is composed of 2x12 joists spaced at 16 in on center and spanning up to 9 feet along with ¾ in T&G plywood sheathing. The house is supported on a slab on grade. The roof diaphragm is built with composite shingle felt and 15/32 in OSB sheathing. The ceiling is made of ½ in thick gypsum wallboard.

4.2 Description of Construction Variants

Three construction variants are defined for the large-house index building. The variants are representative of poor-quality, typical-quality, and superior-quality construction, respectively. The characteristics of each construction variant are described below. These characteristics are summarized in Table 4.1 (Porter, 2001).

Table 4.1 Summaries of Construction Variants for Large-House Index Building.

Superior Quality	Typical Quality	Poor Quality
Good nailing of diaphragms 100% of stiffness and strength from high-quality laboratory tests	Average nailing of diaphragms 90% of stiffness and strength from high-quality laboratory tests	Poor nailing of diaphragms 80% of stiffness and strength from high-quality laboratory tests
Good nailing of shear walls 100% of stiffness and strength from high-quality laboratory tests	Average nailing of shear walls 5% greater of nail spacing	Poor nailing of shear walls 20% greater of nail spacing 5% reduction of stiffness and strength due to water damage
Good connections between structural elements 100% of stiffness and strength from high-quality laboratory tests	Typical connections between structural elements 10% reduction of stiffness and strength in shear walls from high-quality laboratory tests	Poor connections between structural elements 20% reduction of stiffness and strength in shear walls from high-quality laboratory tests
Good Quality Stucco 100% of stiffness and strength from high-quality laboratory tests	Average Quality Stucco 90% of stiffness and strength from high-quality laboratory tests	Poor-Quality Stucco 70% of stiffness and strength from high-quality laboratory tests
Superior Nailing of interior gypsum wallboard 100% of stiffness and strength from high-quality laboratory tests	Good Nailing of interior gypsum wallboard 85% of stiffness and strength from high-quality laboratory tests	Poor Nailing of interior gypsum wallboard 75% of stiffness and strength from high-quality laboratory tests

4.2.1 Characteristics of Poor-Quality Variant

1. **Poor nailing of shear walls and diaphragms.** Diaphragm nail spacing too large in numerous locations; splitting of rim joists and blocking is common, and there are numerous shiners throughout, resulting in 20% reduction in diaphragm stiffness. Likewise, shear wall nail spacing is too large in numerous locations, say 20% greater than nominal on average. Water damage has occurred at numerous locations, leading to an additional 10% reduction in the strength and stiffness of exterior shear walls within 12" of the footing, resulting in 5% reduction in exterior shear walls.
2. **Poor connections between structural elements.** A35 clips omitted in numerous locations, say 20% of locations. ST6624s between posts and glulams omitted. Holddowns missing or poorly installed (including oversize holes) at 40% of locations, resulting in a 15-20% reduction in shear-wall stiffness on the first floor.
3. **Poor quality stucco.** The stucco thickness is less than that shown on the drawings, and many of the staples connecting the woven wire to the studs are missing or poorly installed. As a consequence, the strength of the exterior stucco wall is 65% to 75% that which would be observed in high-quality laboratory tests.
4. **Poor nailing of gypsum wallboard.** Many missing nails to studs, sill, and top plates. The gypsum wallboard is nailed to the studs with a mixture of gypsum wallboard nails and common nails, with many nails (particularly the common nails) over-driven. The result for modeling purposes is that the strength of the wallboard partitions is 75% of what would be observed in a laboratory test of similar thickness wallboard nailed with gypsum wallboard nails at 7-in centers.

4.2.2 Characteristics of Typical-Quality variant

1. **Average nailing of shearwalls and diaphragms.** Diaphragm nail omitted in a few locations, there is some splitting of rim joists and blocking, and some shiners. The resulting nail spacing is 5% greater than nominal on average. Shear wall nailing exceeds maximum spacing at some locations, resulting in nail spacing say 5% greater than nominal on average.
2. **Average connections between structural elements.** A35 clips omitted in some locations, say 5% of locations. ST6224s between posts and glulams installed with some nails missing, reducing strength by say 10%. Holddowns poorly installed at some locations, say 10% of locations.
3. **Average quality stucco.** Some water damage has occurred, and staple spacing exceeds maximum at some locations. As a consequence, the strength of the exterior stucco wall is 90% that which would be observed in high-quality laboratory tests.
4. **Average nailing of gypsum wallboard.** Some missing nails to studs, sill, and top plates. Few nails are over-driven. The strength of the wall is 80% to 90% of laboratory test results for similar construction.

4.2.3 Characteristics of Superior-Quality variant

1. **Good nailing of shear walls and diaphragms.** Diaphragm and shear-wall nailing exceeds requirements, resulting in strength equal to that found in high-quality laboratory tests.

2. **Good connections between structural elements.** All connections installed properly, with strength equal to that shown in high-quality laboratory tests.
3. **Good quality stucco.** The stucco thickness is as shown on the drawings. The woven wire is fixed to each stud by staples as shown in the drawings. The woven wire is stapled to the sill plate and the top plate at 6” centers. No pre-existing deterioration exists. Strength approximately matches that exhibited by high-quality laboratory tests.
4. **Good nailing of gypsum wallboard.** No missing or over-driven nails to studs, sill, or top plates. The strength of the gypsum wallboard partitions matches that of high-quality laboratory tests.

4.3 Modeling Assumptions

Each stucco wall, OSB shear wall and gypsum wall is modeled by an in-plane shear element exhibiting the Wayne Stewart hysteresis rule, as described in Chapter 2. All exterior walls are sheathed with stucco and OSB on the outside and gypsum on the inside. Three parallel shear elements are used to simulate the in-plane behavior of the three sheathing materials. Interior walls consist of gypsum wallboards and, in some cases, OSB shear walls. For these combined OSB and gypsum walls, two parallel shear elements are used as for the exterior walls. For the wall sheathed with gypsum on both sides, only one shear element per wall line is considered. The in-plane behavior of the floor diaphragm is modeled by linear-elastic quadrilateral finite elements. The frame elements used previously in the model of the small-house index building were not used in the model of the large-house building. The connection between the corners of the quadrilateral elements and the shear wall elements was accomplished by constraining the in-plane displacement of the top node of a shear wall to be the same as the displacement of the corners nodes of each quadrilateral elements it connects into. For example, the displacements in the y-direction of nodes 14, 23, 32, 41, 50, and 59 are constrained to be the same as the displacement in the y-direction of node 11.

4.4 Node Numbering

The pancake model of the large-house index building incorporates three layers of nodes, as illustrated in Figure 4.2. The eleven first floor nodes are located below the walls on the first story of the building. The earthquake ground motion is applied simultaneously at these nodes. The second floor nodes are located at the level of the second floor diaphragm and are used to connect the shear elements representing the walls on the first floor from the first floor level to the second floor diaphragm and to connect also the shear elements representing the vertical walls between the second floor diaphragm and the ceiling level. The ceiling nodes are used to connect the interior and exterior shear elements representing the vertical walls on the second floor.

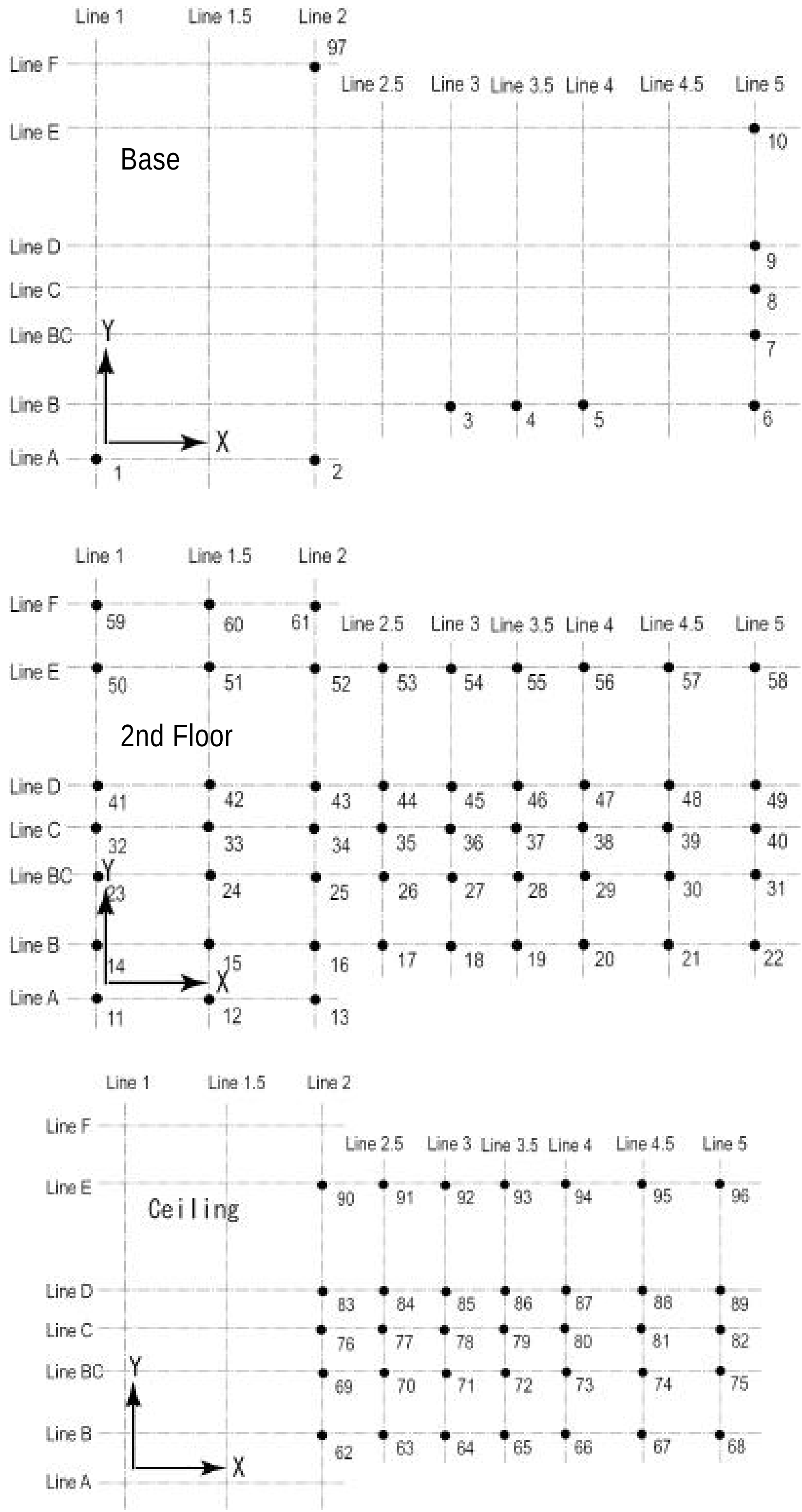


Figure 4.2 Node Numbering for Pancake Model of Large-House Index Building.

4.5 Elements Description and Location

The various elements used to represent the lateral load resisting system of the large-house index building are briefly described in this section.

Twenty-five shear elements are used to represent the walls on the first floor of the large-house index building. Figure 4.3 shows the location, orientation and number assigned to each of these shear elements in the RUAUMOKO data files.

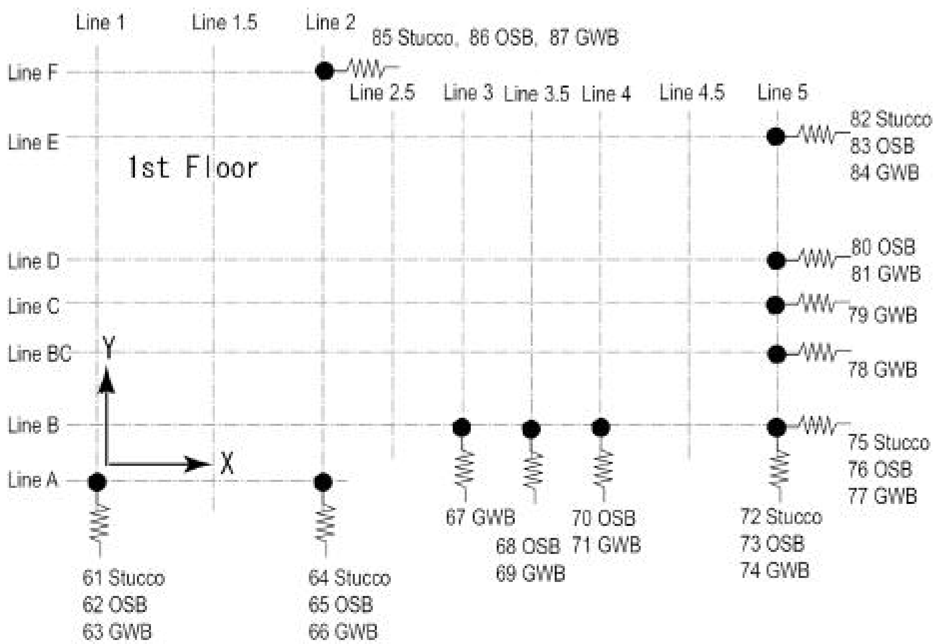


Figure 4.3 Locations, Orientation and Numbering of Shear Elements for Interior and Exterior Vertical Walls on the First floor of the Large-House Index Building.

The location, orientation and number assigned to the twenty-one shear elements used to represent the walls on the second floor of the large-house index building are illustrated in Figure 4.4.

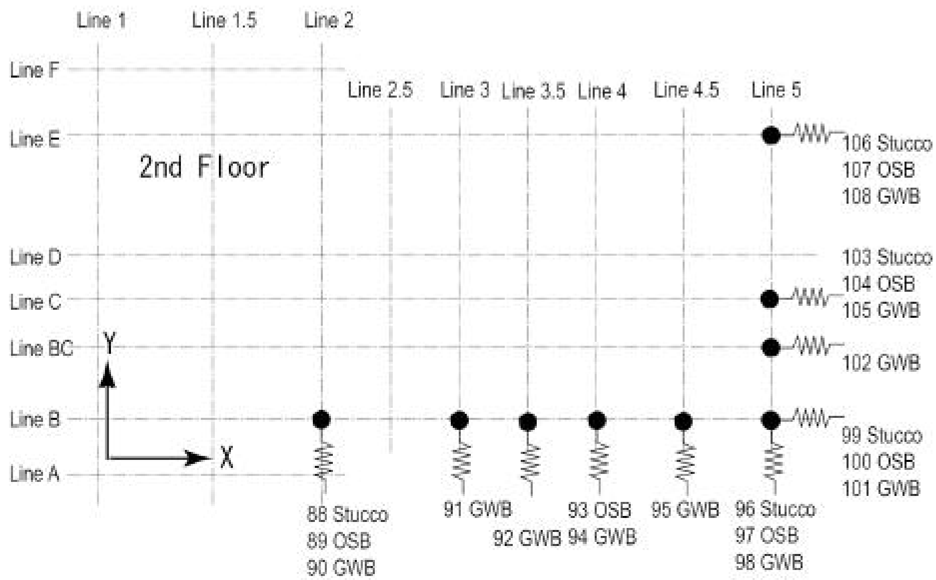


Figure 4.4 Locations, Orientation and Numbering of Shear Elements for Interior and Exterior Vertical Walls on the second floor of the Large-House Index Building.

The location, orientation and number assigned to linear-elastic quadrilateral finite elements used to represent the floor and ceiling diaphragms of the large-house index building are illustrated in Figure 4.5 and Figure 4.6.

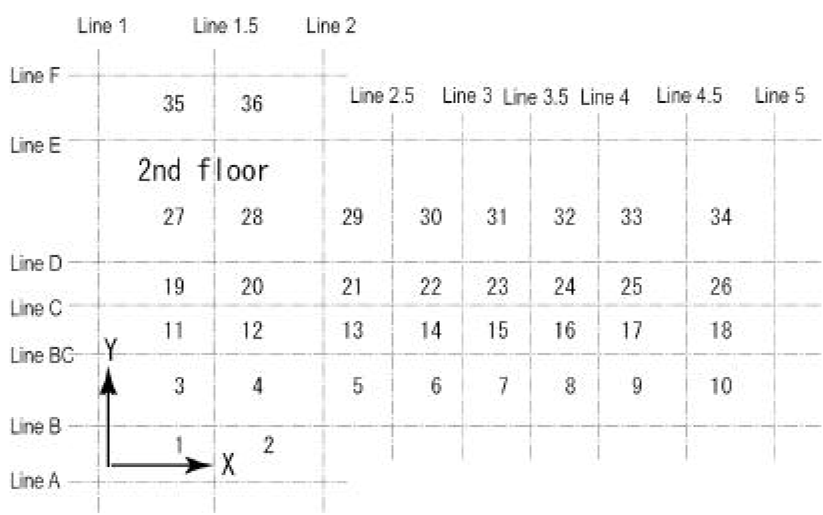


Figure 4.5 Locations, Orientation and Numbering of Quadrilateral Elements for the Second Floor of the Large-House Index Building.

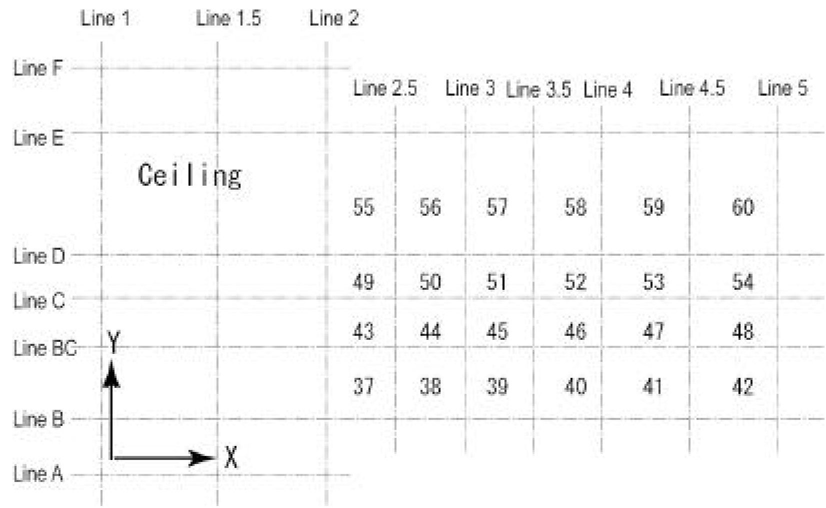


Figure 4.6 Locations, Orientation and Numbering of Quadrilateral Elements for the Ceiling of the Large-House Index Building.

4.6 Hysteretic Parameters for Vertical Wall Shear Elements

The in-plane cyclic responses of the vertical walls incorporated in the large-house index building were modeled by shear elements exhibiting the Wayne Stewart hysteresis rule, described in Chapter 2. The hysteretic parameters for each stucco and gypsum wall were estimated from available cyclic test data on wall assemblies. The hysteretic parameters for the OSB shear walls were computed by the computer program CASHEW: Cyclic Analysis of wood SHEar Walls (Folz and Filiatrault, 2000) developed under Task 1.5.1 of the CUREE-Caltech Woodframe Project.

Figure 4.7 shows the hysteresis loops obtained from one of three cyclic tests on 8 ft x 8 ft walls sheathed with 7/8-in thick stucco on one side. These tests were conducted at the University of California at Irvine for the City of Los Angeles (COLA) Project (Pardoen, 2000). The wire mesh was attached to the wood framing by 1-in crown staples spaced at 6 in on center along the vertical studs (spaced at 16 in on center) and at 6 in along the top and bottom plate. Table 4.2 shows the resulting parameters of the Wayne Stewart hysteresis rule extracted from these test results. The hysteretic properties assigned to the shear elements representing the exterior vertical stucco walls were obtained by adjusting the strength and stiffness values of Table 4.2 for the actual length of full wall piers in each wall line.

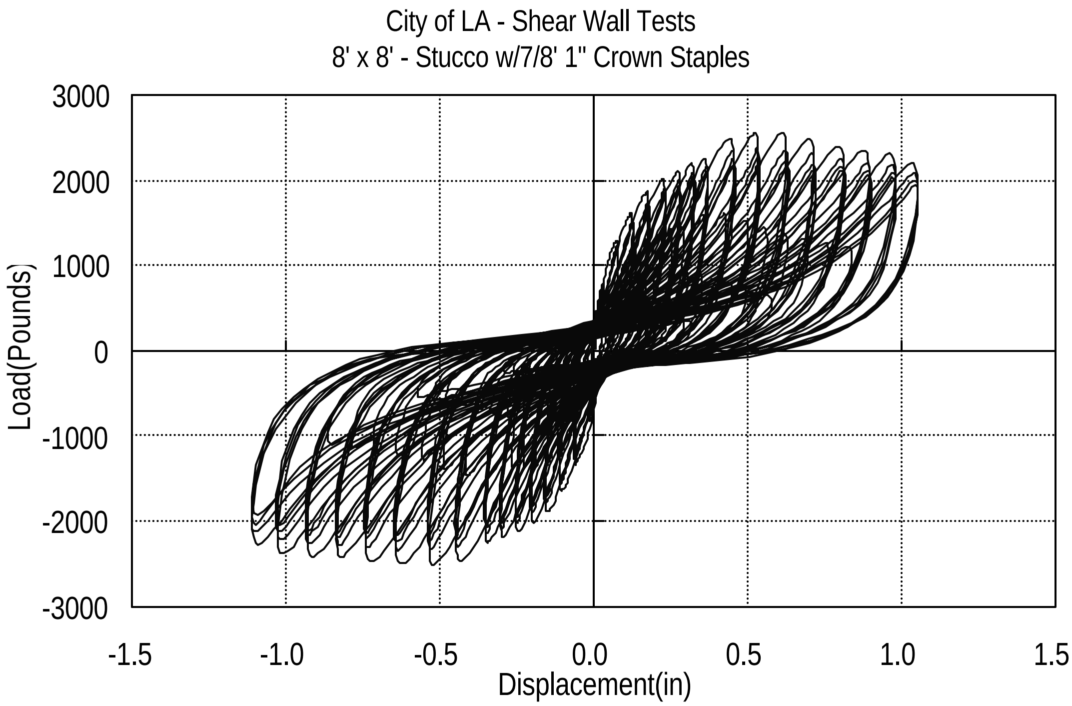


Figure 4.7 Hysteresis Loops From Cyclic Test on 8 ft x 8 ft Wall Sheathed with 7/8-in Thick Stucco on One Side (from Pardoen, 2000).

The gypsum walls had the same property as that of the small-house index building. Their parameters were slightly modified, however, to fit to the experimental results more closely. The hysteretic properties assigned to the shear elements representing the interior vertical gypsum walls were obtained by adjusting the strength and stiffness values of Table 4.2 for the actual length of full wall piers in each wall line. The same procedure was adopted for the interior gypsum sheathing of the exterior walls, but the values were divided by two since these walls are sheathed with gypsum wallboard only on one side.

The properties of the OSB shear walls were predicted by the computer program CASHEW: Cyclic Analysis of wood SHEar Walls (Folz and Filiatrault, 2000). The hysteretic parameters of the sheathing-to-framing connectors obtained from the cyclic loading tests carried under Task 1.1.1 of the CUREE-Caltech Woodframe Project are shown in Table 4.3 and were used as basic input into the CASHEW program. An example of an analyzed shear wall is shown in Figure 4.8. A nail edge spacing of ½-inch was assumed at each corner. In the cyclic analysis of the poor-quality and typical-quality variants, there were some walls that were unable to be run with the CASHEW program. For those cases, the backbone parameters such as *Fu*, *Fy*, *FI*, K_0 were obtained from the monotonic analysis results and the other parameters were assumed to be the same as the superior quality. The resulting properties are shown in Tables 4.4, 4.5, and 4.6 for the poor-quality, typical-quality and superior-quality variant, respectively.

Table 4.2 Parameters of Wayne Stewart Hysteresis Rule Obtained from Experimental Results

Wall Type	Length (in)	Sheathed Sides	F_y (kips)	k_o (kips/in)	R_f	F_u (kips)	F_i (kips)	PTRI
Vertical Stucco	96	1	1.65	27.9	0.065	2.61	0.49	-0.023
Vertical Gypsum	96	2	2.54	38.0	0.063	3.42	0.58	-0.041
PUNL = 1.45, $a_1 = 0.38$ and $b_1 = 1.09$ for all wall types								

Table 4.3 Sheathing-to Framing Connector Parameters

k_o (kips/in)	r_1	r_2	r_3	r_4	F_0 (kips)	F_i (kips)	D_u (in)	a_1	b_1
4.87	0.049	-0.049	1.40	0.015	0.18	0.042	0.50	0.80	1.10

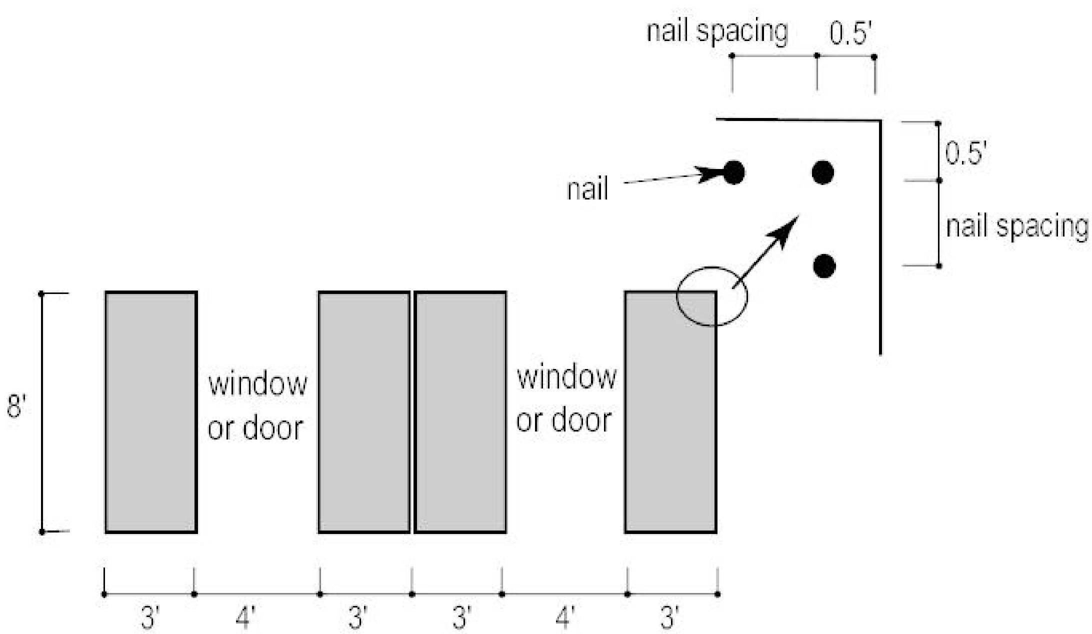


Figure 4.8 OSB Shearwall Model for Line F on the First Floor of the Large-House Index Building.

Table 4.4 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Poor-Quality Variant of the Large-House Index Building.

Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Exterior Stucco	1st	Line 1	61	5.11	86.77	0.066	8.12	1.28	-0.024	1.45	0.38	1.09
		Line 2	64	1.59	26.89	0.066	2.52	0.40	-0.024	1.45	0.38	1.09
		Line 5	72	0.87	14.67	0.066	1.38	0.22	-0.024	1.45	0.38	1.09
		Line B	75	3.03	51.33	0.066	4.81	0.76	-0.024	1.45	0.38	1.09
		Line E	82	1.80	30.56	0.066	2.86	0.45	-0.024	1.45	0.38	1.09
		Line F	85	1.73	29.33	0.066	2.75	0.44	-0.024	1.45	0.38	1.09
	2nd	Line 2	88	3.60	61.11	0.066	5.72	0.90	-0.024	1.45	0.38	1.09
		Line 5	96	2.31	39.11	0.066	3.66	0.58	-0.024	1.45	0.38	1.09
		Line B	99	4.47	75.77	0.066	7.09	1.12	-0.024	1.45	0.38	1.09
		Line C	103	0.72	12.23	0.066	1.15	0.18	-0.024	1.45	0.38	1.09
		Line E	106	2.88	48.89	0.066	4.58	0.72	-0.024	1.45	0.38	1.09
Vertical shear wall OSB	1st	Line 1	62	7.87	25.89	0.071	12.39	1.92	-0.048	1.18	0.76	1.10
		Line 2	65	9.00	29.95	0.073	14.17	2.20	-0.062	1.16	0.76	1.10
		Line 35	68	1.96	6.50	0.071	3.09	0.48	-0.053	1.17	0.76	1.10
		Line 4	70	4.01	10.95	0.076	6.31	0.98	-0.061	1.08	0.76	1.09
		Line 5	73	2.61	8.64	0.069	4.11	0.64	-0.050	1.19	0.76	1.10
		Line B	76	6.48	21.66	0.073	10.20	1.58	-0.059	1.19	0.75	1.10
		Line D	80	3.97	12.96	0.071	6.25	0.97	-0.047	1.18	0.77	1.10
		Line E	83	5.62	17.12	0.066	8.84	1.37	-0.055	1.20	0.78	1.09
		Line F	86	2.85	9.08	0.066	4.48	0.70	-0.056	1.23	0.76	1.10
	2nd	Line 2	89	5.40	18.05	0.075	8.50	1.32	-0.061	1.17	0.75	1.09
		Line 4	93	3.89	12.98	0.071	6.12	0.95	-0.055	1.17	0.75	1.09
		Line 5	97	4.91	16.19	0.070	7.72	1.20	-0.049	1.18	0.76	1.10
		Line B	100	9.14	25.95	0.073	14.38	2.23	-0.051	1.17	0.75	1.10
		Line C	104	1.17	3.81	0.073	1.84	0.29	-0.067	1.16	0.75	1.09
		Line E	107	5.86	16.19	0.071	9.23	1.43	-0.040	1.19	0.76	1.10
Vertical Gypsum	1st	Line 1	63	4.21	63.24	0.064	5.70	1.06	-0.042	1.45	0.38	1.09
		Line 2	66	3.55	53.44	0.064	4.82	0.89	-0.042	1.45	0.38	1.09
		Line 3	67	2.37	35.63	0.064	3.22	0.60	-0.042	1.45	0.38	1.09
		Line 35	69	0.77	11.58	0.064	1.05	0.20	-0.042	1.45	0.38	1.09
		Line 4	71	0.77	11.58	0.064	1.05	0.20	-0.042	1.45	0.38	1.09
		Line 5	74	0.71	10.69	0.064	0.97	0.18	-0.042	1.45	0.38	1.09
		Line B	77	5.33	80.16	0.064	7.23	1.34	-0.042	1.45	0.38	1.09
		Line BC	78	1.90	28.50	0.064	2.57	0.48	-0.042	1.45	0.38	1.09
		Line C	79	1.90	28.50	0.064	2.57	0.48	-0.042	1.45	0.38	1.09
		Line D	81	1.78	26.72	0.064	2.41	0.45	-0.042	1.45	0.38	1.09
		Line E	84	1.48	22.27	0.064	2.01	0.37	-0.042	1.45	0.38	1.09
		Line F	87	1.42	21.38	0.064	1.93	0.36	-0.042	1.45	0.38	1.09
	2nd	Line 2	90	2.96	44.54	0.064	4.02	0.74	-0.042	1.45	0.38	1.09
		Line 3	91	4.97	74.82	0.064	6.75	1.25	-0.042	1.45	0.38	1.09
		Line 3.5	92	1.42	21.38	0.064	1.93	0.36	-0.042	1.45	0.38	1.09
		Line 4	94	2.61	39.19	0.064	3.54	0.66	-0.042	1.45	0.38	1.09
		Line 4.5	95	1.42	21.38	0.064	1.93	0.36	-0.042	1.45	0.38	1.09
		Line 5	98	1.90	28.50	0.064	2.57	0.48	-0.042	1.45	0.38	1.09
		Line B	101	6.51	97.97	0.064	8.84	1.63	-0.042	1.45	0.38	1.09
		Line BC	102	4.74	71.25	0.064	6.43	1.19	-0.042	1.45	0.38	1.09
		Line C	105	6.28	94.41	0.064	8.51	1.57	-0.042	1.45	0.38	1.09
		Line E	108	2.37	35.63	0.064	3.22	0.60	-0.042	1.45	0.38	1.09

Table 4.5 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Typical-Quality Variant of the Large-House Index Building.

Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Exterior Stucco	1st	Line 1	61	6.57	111.56	0.066	10.44	1.65	-0.024	1.45	0.38	1.09
		Line 2	64	2.04	34.57	0.066	3.24	0.51	-0.024	1.45	0.38	1.09
		Line 5	72	1.12	18.86	0.066	1.77	0.28	-0.024	1.45	0.38	1.09
		Line B	75	3.89	65.99	0.066	6.18	0.98	-0.024	1.45	0.38	1.09
		Line E	82	2.32	39.28	0.066	3.68	0.58	-0.024	1.45	0.38	1.09
		Line F	85	2.23	37.71	0.066	3.53	0.56	-0.024	1.45	0.38	1.09
	2nd	Line 2	88	4.63	78.56	0.066	7.35	1.16	-0.024	1.45	0.38	1.09
		Line 5	96	2.97	50.28	0.066	4.71	0.75	-0.024	1.45	0.38	1.09
		Line B	99	5.74	97.42	0.066	9.12	1.44	-0.024	1.45	0.38	1.09
		Line C	103	0.93	15.72	0.066	1.47	0.24	-0.024	1.45	0.38	1.09
		Line E	106	3.71	62.85	0.066	5.88	0.93	-0.024	1.45	0.38	1.09
Vertical shear wall OSB	1st	Line 1	62	10.76	38.06	0.071	17.12	2.62	-0.050	1.18	0.76	1.10
		Line 2	65	12.12	44.21	0.072	19.65	2.97	-0.062	1.17	0.76	1.10
		Line 35	68	2.91	8.73	0.064	4.69	0.71	-0.058	1.17	0.76	1.10
		Line 4	70	5.48	13.77	0.064	8.84	1.33	-0.058	1.17	0.76	1.10
		Line 5	73	4.33	13.69	0.064	6.98	1.05	-0.058	1.17	0.76	1.10
		Line B	76	8.66	32.07	0.072	14.15	2.12	-0.066	1.16	0.75	1.09
		Line D	80	5.49	18.96	0.071	8.65	1.34	-0.048	1.18	0.77	1.11
		Line E	83	7.92	26.37	0.074	12.69	1.91	-0.060	1.11	0.76	1.09
		Line F	86	3.76	12.88	0.064	6.21	0.89	-0.066	1.18	0.77	1.10
	2nd	Line 3	89	7.22	26.73	0.072	11.79	1.77	-0.066	1.16	0.75	1.09
		Line 4	93	5.28	19.22	0.071	8.50	1.29	-0.055	1.17	0.76	1.09
		Line 5	97	6.81	23.88	0.069	10.71	1.65	-0.043	1.18	0.77	1.10
		Line B	100	10.56	38.44	0.071	16.99	2.57	-0.055	1.17	0.76	1.09
		Line C	104	1.45	5.35	0.072	2.36	0.36	-0.066	1.16	0.75	1.09
		Line E	107	6.64	23.01	0.064	10.71	1.62	-0.058	1.17	0.76	1.10
Vertical Gypsum	1st	Line 1	63	4.77	71.67	0.064	6.46	1.20	-0.042	1.45	0.38	1.09
		Line 2	66	4.03	60.57	0.064	5.46	1.01	-0.042	1.45	0.38	1.09
		Line 3	67	2.69	40.38	0.064	3.64	0.68	-0.042	1.45	0.38	1.09
		Line 35	69	0.88	13.13	0.064	1.19	0.22	-0.042	1.45	0.38	1.09
		Line 4	71	0.88	13.13	0.064	1.19	0.22	-0.042	1.45	0.38	1.09
		Line 5	74	0.81	12.12	0.064	1.10	0.21	-0.042	1.45	0.38	1.09
		Line B	77	6.04	90.85	0.064	8.19	1.51	-0.042	1.45	0.38	1.09
		Line BC	78	2.15	32.30	0.064	2.92	0.54	-0.042	1.45	0.38	1.09
		Line C	79	2.15	32.30	0.064	2.92	0.54	-0.042	1.45	0.38	1.09
		Line D	81	2.02	30.29	0.064	2.73	0.51	-0.042	1.45	0.38	1.09
		Line E	84	1.68	25.24	0.064	2.28	0.42	-0.042	1.45	0.38	1.09
		Line F	87	1.61	24.23	0.064	2.19	0.41	-0.042	1.45	0.38	1.09
	2nd	Line 3	90	3.36	50.47	0.064	4.55	0.84	-0.042	1.45	0.38	1.09
		Line 3	91	5.64	84.79	0.064	7.65	1.41	-0.042	1.45	0.38	1.09
		Line 3.5	92	1.61	24.23	0.064	2.19	0.41	-0.042	1.45	0.38	1.09
		Line 4	94	2.95	44.42	0.064	4.01	0.74	-0.042	1.45	0.38	1.09
		Line 4.5	95	1.61	24.23	0.064	2.19	0.41	-0.042	1.45	0.38	1.09
		Line 5	98	2.15	32.30	0.064	2.92	0.54	-0.042	1.45	0.38	1.09
		Line B	101	7.38	111.04	0.064	10.01	1.85	-0.042	1.45	0.38	1.09
		Line BC	102	5.37	80.75	0.064	7.28	1.35	-0.042	1.45	0.38	1.09
		Line C	105	7.11	107.00	0.064	9.65	1.78	-0.042	1.45	0.38	1.09
		Line E	108	2.69	40.38	0.064	3.64	0.68	-0.042	1.45	0.38	1.09

Table 4.6 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Superior-Quality Variant of the Large-House Index Building.

Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Exterior Stucco	1st	Line 1	61	7.30	123.95	0.066	11.60	1.83	-0.024	1.45	0.38	1.09
		Line 2	64	2.27	38.41	0.066	3.60	0.57	-0.024	1.45	0.38	1.09
		Line 5	72	1.24	20.95	0.066	1.96	0.31	-0.024	1.45	0.38	1.09
		Line B	75	4.32	73.33	0.066	6.86	1.08	-0.024	1.45	0.38	1.09
		Line E	82	2.58	43.65	0.066	4.09	0.65	-0.024	1.45	0.38	1.09
		Line F	85	2.47	41.90	0.066	3.92	0.62	-0.024	1.45	0.38	1.09
	2nd	Line 2	88	5.15	87.29	0.066	8.17	1.29	-0.024	1.45	0.38	1.09
		Line 5	96	3.29	55.87	0.066	5.23	0.83	-0.024	1.45	0.38	1.09
		Line B	99	6.38	108.24	0.066	10.13	1.60	-0.024	1.45	0.38	1.09
		Line C	103	1.03	17.46	0.066	1.64	0.26	-0.024	1.45	0.38	1.09
		Line E	106	4.12	69.83	0.066	6.54	1.03	-0.024	1.45	0.38	1.09
Vertical shear wall OSB	1st	Line 1	62	12.79	44.31	0.071	20.33	3.12	-0.050	1.17	0.76	1.10
		Line 2	65	14.41	51.41	0.073	23.50	3.53	-0.063	1.16	0.76	1.10
		Line 35	68	3.26	11.48	0.072	5.23	0.80	-0.051	1.16	0.75	1.09
		Line 4	70	6.12	15.93	0.068	9.84	1.49	-0.051	1.13	0.73	1.06
		Line 5	73	4.26	13.53	0.071	6.85	1.04	-0.056	1.17	0.76	1.09
		Line B	76	10.30	37.27	0.073	16.81	2.53	-0.067	1.16	0.75	1.09
		Line D	80	6.53	22.08	0.071	10.28	1.59	-0.049	1.18	0.76	1.10
		Line E	83	9.55	30.85	0.075	15.29	2.30	-0.063	1.10	0.76	1.09
		Line F	86	4.44	15.05	0.064	7.33	1.06	-0.067	1.17	0.77	1.10
	2nd	Line 3	89	8.58	31.06	0.073	14.01	2.11	-0.068	1.16	0.75	1.09
		Line 4	93	6.28	22.36	0.072	10.09	1.53	-0.055	1.16	0.75	1.09
		Line 5	97	8.09	27.81	0.070	12.71	1.95	-0.044	1.17	0.76	1.10
		Line B	100	12.54	44.73	0.072	20.18	3.05	-0.055	1.16	0.75	1.09
		Line C	104	1.72	6.21	0.073	2.81	0.43	-0.068	1.16	0.75	1.09
		Line E	107	8.09	27.81	0.070	12.71	1.95	-0.044	1.17	0.76	1.10
Vertical Gypsum	1st	Line 1	63	5.61	84.32	0.064	7.60	1.41	-0.042	1.45	0.38	1.09
		Line 2	66	4.74	71.25	0.064	6.43	1.19	-0.042	1.45	0.38	1.09
		Line 3	67	3.16	47.50	0.064	4.29	0.79	-0.042	1.45	0.38	1.09
		Line 35	69	1.03	15.44	0.064	1.40	0.26	-0.042	1.45	0.38	1.09
		Line 4	71	1.03	15.44	0.064	1.40	0.26	-0.042	1.45	0.38	1.09
		Line 5	74	0.95	14.25	0.064	1.29	0.24	-0.042	1.45	0.38	1.09
		Line B	77	7.10	106.88	0.064	9.64	1.78	-0.042	1.45	0.38	1.09
		Line BC	78	2.53	38.00	0.064	3.43	0.64	-0.042	1.45	0.38	1.09
		Line C	79	2.53	38.00	0.064	3.43	0.64	-0.042	1.45	0.38	1.09
		Line D	81	2.37	35.63	0.064	3.22	0.60	-0.042	1.45	0.38	1.09
		Line E	84	1.98	29.69	0.064	2.68	0.50	-0.042	1.45	0.38	1.09
		Line F	87	1.90	28.50	0.064	2.57	0.48	-0.042	1.45	0.38	1.09
	2nd	Line 3	90	3.95	59.38	0.064	5.36	0.99	-0.042	1.45	0.38	1.09
		Line 3	91	6.63	99.75	0.064	9.00	1.66	-0.042	1.45	0.38	1.09
		Line 3.5	92	1.90	28.50	0.064	2.57	0.48	-0.042	1.45	0.38	1.09
		Line 4	94	3.48	52.25	0.064	4.71	0.87	-0.042	1.45	0.38	1.09
		Line 4.5	95	1.90	28.50	0.064	2.57	0.48	-0.042	1.45	0.38	1.09
		Line 5	98	2.53	38.00	0.064	3.43	0.64	-0.042	1.45	0.38	1.09
		Line B	101	8.68	130.63	0.064	11.78	2.17	-0.042	1.45	0.38	1.09
		Line BC	102	6.31	95.00	0.064	8.57	1.58	-0.042	1.45	0.38	1.09
		Line C	105	8.37	125.88	0.064	11.35	2.10	-0.042	1.45	0.38	1.09
		Line E	108	3.16	47.50	0.064	4.29	0.79	-0.042	1.45	0.38	1.09

4.7 Properties of Horizontal Floor and Roof Diaphragms

The in-plane stiffness of the floor diaphragm, G_d, is taken to be 800 kips/in, while the corresponding value for the roof diaphragm is take to be 400 kips/in. These values are prescribed by the NEHRP Guidelines for Seismic Rehabilitation of Buildings (FEMA, 1997). Each linear-elastic diaphragm finite element is assigned elastic properties such that:

$$Gt = \frac{Et}{2(1+\boldsymbol{n})} = G_d$$

where G is the equivalent shear modulus, E is the equivalent elastic modulus, $\boldsymbol{n}$ is the equivalent Poisson's ratio and t is the thickness of the finite element.

4.8 Weight Distribution

Table 4.7 lists the weights considered for the large-house index building. These weights were distributed as nodal lumped seismic weights according to the tributary areas of the nodes (see Figure 4.2).

4.9 Description of RUAUMOKO Data Files

The three RUAMOKO data files corresponding to the poor-quality, typical-quality and superior-quality variants of the large-house index building are included in the CD-ROM accompanying this report. These data files are self-contained and include, as ground motion input, one component of the acceleration time-history recorded at Canoga Park during the 1994 Northridge Earthquake. This ground motion is oriented along the short side of the building.

4.10 Analysis Examples

In this section, the three RUAUMOKO data files are used to evaluate the seismic response of the three variants of the large-house building index when excited parallel to the short side of the building (y-axis direction) by the Canoga Park record of the 1994 Northridge Earthquake scale to a Peak Ground Acceleration (PGA) of 0.50 g.

Table 4.7 Weights for Large-House Index Building.

Item	Location		Length (ft)	Width or Height (ft)	Area (sq ft)	Unit Weight (psf)	Weight #	Total Weight #
Main Roof / 2-6			40	26	1040	14	14,560	14,560
Main Ceiling / 2-6			40	26	1040	4.8	4,992	4,992
2nd Floor Walls	Line B-Ext	Total	40	8	320			4,035
		window/door			30	4	120	
		typ exterior			290	13.5	3,915	
	Line B.2-Int	Total	0	0	0			0
		open & door			0	2	0	
		typ interior			0	5	0	
	Line B.6-Int	Total	25	8	200			901
		open & door			33	2	66	
		typ interior			167	5	835	
	Line C-Int	Total	36	8	288			1,290
		open & door			50	2	100	
		typ interior			238	5	1,190	
	Line E-Ext	Total	40	8	320			3,256
		window/door			112	4	448	
		typ exterior			208	13.5	2,808	
	Line 2-Ext	Total	26	6	156			2,512
		window/door			12	4	48	
		typ ext			144	13.5	1,944	
		int-garage			104	5	520	
	Line 2.8-Int	Total	11.5	8	92			259
		open & door			67	2	134	
		typ interior			25	5	125	
	Line 3-Int	Total	24	8	192			960
		open & door			0	2	0	
		typ interior			192	5	960	
	Line 3.2-Int	Total	11.5	8	92			259
		open & door			67	2	134	
		typ interior			25	5	125	
	Line 3.7-Int	Total	6	8	48			240
		open & door			0	2	0	
		typ interior			48	5	240	
	Line 4-Int	Total	19	10.5	199.5			1,222
		open & door			16.7	2	33	
		interior w/ply			182.8	6.5	1,188	
	Line 4.3-Int	Total	6	10.5	63			265
		open & door			16.7	2	33	
		typ interior			46.3	5	232	
	Line 4.7-Int	Total	10	8	80			350
		open & door			16.7	2	33	

Table 4.7 Weights for Large-House Index Building (continued).

Item	Location		Length (ft)	Width or Height (ft)	Area (sq ft)	Unit Weight (psf)	Weight #	Total Weight #
	Line 5-Ext	Total	16	10.5	168			720
		open & door			40	2	80	
		typ interior			128	5	640	
	Line 6-Ext	Total	10	9.5	95			475
		closet door			0	1	0	
		typ interior			95	5	475	
Main Floor / 2-6			40	32	1280	8.6	11008	11008
Low Roof / 1-2			20	37	740	14	10360	10360
Low Ceiling / 1-2			20	37	740	4.8	3552	3552
1st Floor Walls	Line A-Ext	Total	20	8	160			1,144
		window/door			107	4	428	
		typ exterior			53	13.5	716	
	Line B-Ext	Total	40	8	320			3,598
		open & door			76	4	304	
		typ exterior			244	13.5	3,294	
	Line B.2-Int	Total	12	8	96			480
		open & door			0	2	0	
		typ interior			96	5	480	
	Line B.6-Int	Total	15	8	120			552
		open & door			16	2	32	
		typ interior			104	5	520	
	Line C-Int	Total	19	8	152			760
		open & door			0	2	0	
		typ interior			152	5	760	
	Line D-Int	Total	20	8	160			784
		window/door			16	4	64	
		typ interior			144	5	720	
	Line E-Ext	Total	40	8	320			2,986
		open & door			116	2	232	
		typ exterior			204	13.5	2,754	
	Line F-Ext	Total	20	8	160			1,856
		window/door			32	4	128	
		typ exterior			128	13.5	1,728	
	Line 1-Ext	Total	37	12	444			5,766
		window/door			24	4	96	
		typ ext			420	13.5	5,670	
	Line 2-Int-Ex	Total	26	8	208			2,207
		window/door			16	2	32	
		typ interior			192	5	960	
		exterior			90	13.5	1,215	
	Line 2.7-Int	Total	4	8	32			160
		open & door			0	2	0	
		typ interior			32	5	160	
	Line 3-Int	Total	25	8	200			692
		open & door			77	1	77	

Table 4.7 Weights for Large-House Index Building (continued).

Item	Location		Length (ft)	Width or Height (ft)	Area (sq ft)	Unit Weight (psf)	Weight #	Total Weight #
		typ interior			123	5	615	
	Line 3.5-Int	Total	8	8	64			416
		open & door			0	2	0	
		interior w/ply			64	6.5	416	
	Line 4-Int	Total	8	8	64			416
		open & door			0	2	0	
		typ interior			64	6.5	416	
	Line 5-Ext	Total	26	8	208			2,124
		open & door			72	4	288	
		typ exterior			136	13.5	1,836	
Total Building Weight								85,156

Table 4.8 shows the fundamental frequency computed based on the initial stiffness for each of the construction variant of the large-house index building. Figure 4.9 shows, for the three construction variants the displacement time-histories in the y-direction at the second floor and ceiling level, respectively. Figure 4.10 presents, for the three construction variants, the hysteresis loops of the OSB shearwall along wall line 5 of the index building. The graph on the left hand side represents the behavior of the first floor wall (element 73) while the graph on the right hand side represents the behavior of the second floor wall (element 97). The deformation is concentrated in the first floor shear walls in the three construction variants. For the poor-quality variant, the maximum displacement reaches a drift value of over 1.5%. On the other hand, the drift of the superior quality variant remains within 0.5%.

Table 4.8 Fundamental Frequencies of Large-House Index Building.

Construction Variant	Fundamental Frequency (Hz)	Mode of Vibration
Poor-Quality	5.20	Y-direction
Typical-Quality	5.80	Y-direction
Superior-Quality	6.19	Y-direction

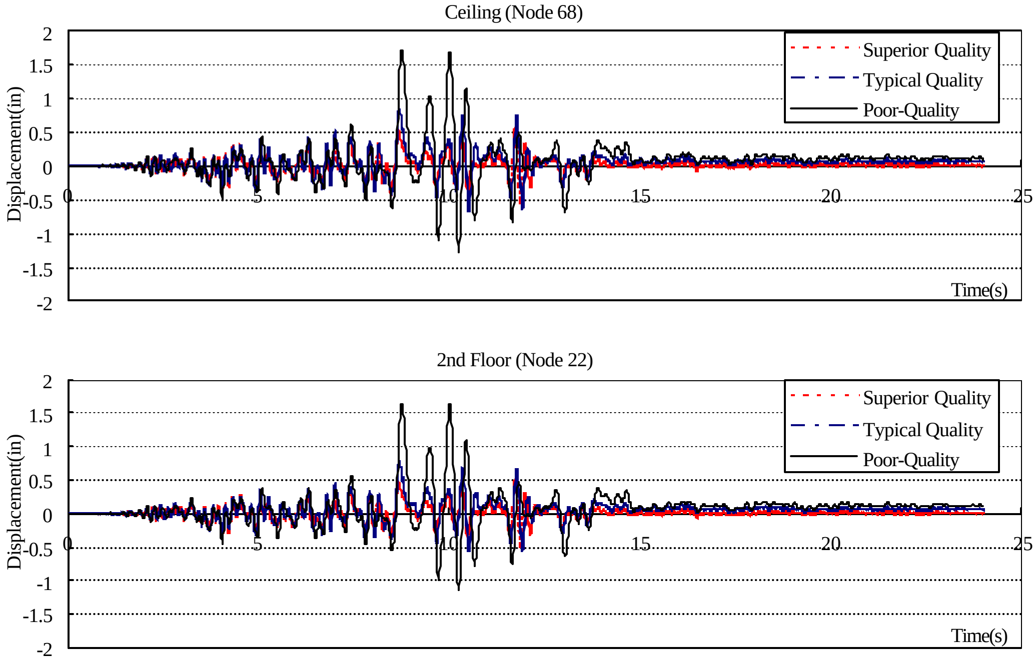


Figure 4.9 Displacement Time-Histories in the Y-Direction for Large-House Index Building Under Canoga Park Record, PGA = 0.50 g.

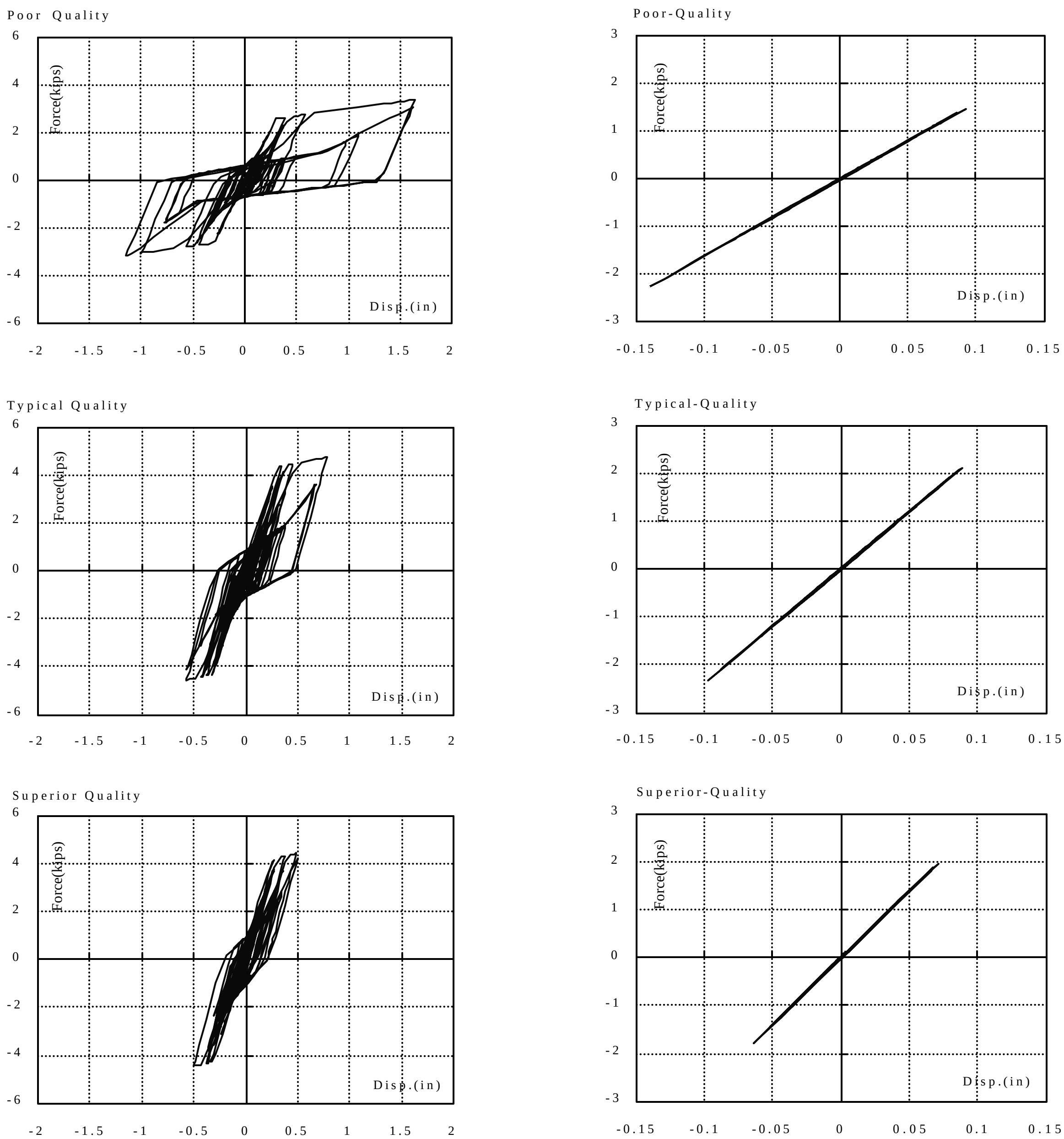


Figure 4.10 Hysteresis Loops of OSB Walls Along Wall Line 5 (Element 73 on the left hand side and 97 on the right hand side) for Large-House Index Building Under Canoga Park Record,

PGA = 0.50 g.

4.11 Retrofit of Large House: Retrofit Measure No. 2

Three retrofit measures were considered for the large-house index building. For the first retrofit measure (Measure No. 2), OSB panels were added above and below all door and window openings, as illustrated in figure 4.11. The resulting properties for the shear elements representing the OSB shear walls retrofitted according to measure No. 2, as computed by the CASHEW program, are shown in Tables 4.9 to 4.11.

The three RUAMOKO data files corresponding to the retrofitted poor-quality, typical-quality and superior-quality variants of the large-house index building are included in the CD-ROM accompanying this report. These data files are self-contained and include, as ground motion input, one component of the acceleration time-history recorded at Canoga Park during the 1994 Northridge Earthquake. This ground motion is oriented along the short side of the building.

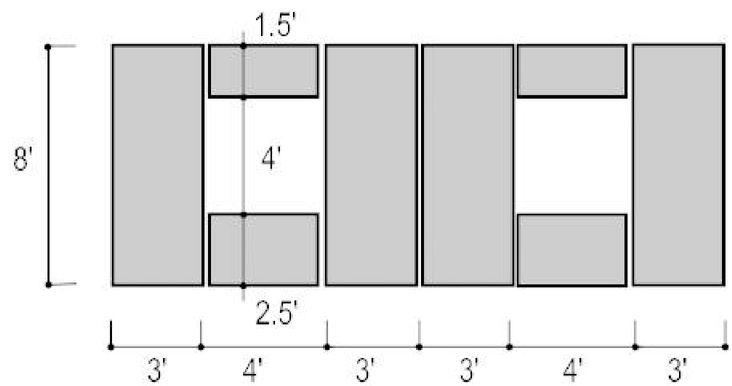


Figure 4.11 CASHEW Model of the OSB shear wall of the first floor Line F of the Large-House Index Building Retrofitted According to Measure No. 2.

Table 4.9 Parameters of Wayne Stewart Hysteresis Rule for the OSB Shear Elements of the Poor-Quality Variant of the Large-House Index Building Retrofitted According to Measure No. 2.

Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical shear wall OSB	1st	Line 1	62	8.06	26.77	0.071	12.86	1.97	-0.048	1.18	0.76	1.10
		Line 2	65	9.43	31.68	0.073	15.04	2.30	-0.062	1.16	0.76	1.10
		Line 35	68	2.04	6.84	0.071	3.25	0.50	-0.053	1.17	0.76	1.10
		Line 4	70	4.17	11.53	0.076	6.65	1.00	-0.061	1.08	0.76	1.09
		Line 5	73	3.12	10.17	0.069	4.98	0.77	-0.050	1.19	0.76	1.10
		Line B	76	7.35	24.73	0.073	11.72	1.80	-0.059	1.19	0.75	1.10
		Line D	80	4.20	13.80	0.071	6.70	1.03	-0.047	1.18	0.77	1.10
		Line E	83	8.10	26.04	0.066	12.19	1.92	-0.055	1.20	0.78	1.09
		Line F	86	3.27	10.82	0.066	5.40	0.81	-0.056	1.23	0.76	1.10
	2nd	Line 2	89	6.11	20.19	0.075	9.74	1.49	-0.061	1.17	0.75	1.09
		Line 4	93	4.04	13.66	0.071	6.45	0.99	-0.055	1.17	0.75	1.09
		Line 5	97	5.21	17.29	0.070	8.30	1.27	-0.049	1.18	0.76	1.10
		Line B	100	9.50	30.68	0.073	15.14	2.32	-0.051	1.17	0.75	1.10
		Line C	104	1.22	4.01	0.073	1.94	0.30	-0.067	1.16	0.75	1.09
		Line E	107	6.09	19.64	0.071	9.71	1.49	-0.040	1.19	0.76	1.10

Table 4.10 Parameters of Wayne Stewart Hysteresis Rule for the OSB Shear Elements of the Typical-Quality Variant of the Large-House Index Building Retrofitted According to Measure No. 2.

Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical shear wall OSB	1st	Line 1	62	11.21	39.24	0.071	17.79	2.74	-0.047	1.18	0.76	1.10
		Line 2	65	12.20	44.36	0.072	19.78	2.99	-0.062	1.17	0.76	1.10
		Line 35	68	2.94	8.73	0.070	4.69	0.72	-0.054	1.18	0.76	1.10
		Line 4	70	5.53	13.77	0.070	8.84	1.36	-0.054	1.18	0.76	1.10
		Line 5	73	4.37	13.69	0.070	6.98	1.07	-0.054	1.18	0.76	1.10
		Line B	76	10.16	36.53	0.073	16.42	2.52	-0.058	1.18	0.76	1.10
		Line D	80	5.57	19.11	0.071	8.78	1.36	-0.048	1.19	0.77	1.11
		Line E	83	10.96	34.36	0.067	16.38	2.58	-0.055	1.18	0.79	1.09
		Line F	86	4.25	13.91	0.066	6.98	1.03	-0.056	1.20	0.77	1.10
	2nd	Line 3	89	7.84	28.00	0.074	12.73	1.95	-0.060	1.18	0.75	1.09
		Line 4	93	5.28	19.22	0.071	8.50	1.29	-0.055	1.17	0.76	1.09
		Line 5	97	6.85	22.07	0.070	10.94	1.68	-0.054	1.18	0.76	1.10
		Line B	100	11.43	40.32	0.072	18.33	2.83	-0.050	1.18	0.76	1.10
		Line C	104	1.45	5.35	0.072	2.36	0.36	-0.066	1.16	0.75	1.09
		Line E	107	7.40	25.42	0.071	11.65	1.83	-0.040	1.20	0.77	1.10

Table 4.11 Parameters of Wayne Stewart Hysteresis Rule for the OSB Shear Elements of the Superior-Quality Variant of the Large-House Index Building Retrofitted According to Measure No. 2.

Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical shear wall OSB	1st	Line 1	62	13.29	45.63	0.072	21.09	3.25	-0.048	1.18	0.76	1.10
		Line 2	65	14.50	51.58	0.073	23.50	3.56	-0.063	1.16	0.76	1.10
		Line 35	68	3.26	11.48	0.072	5.23	0.80	-0.051	1.16	0.75	1.09
		Line 4	70	6.19	15.93	0.068	9.84	1.51	-0.051	1.13	0.73	1.06
		Line 5	73	5.48	17.58	0.068	8.47	1.33	-0.045	1.21	0.77	1.10
		Line B	76	11.97	42.17	0.074	19.31	2.97	-0.060	1.18	0.75	1.10
		Line D	80	6.62	22.25	0.072	10.43	1.62	-0.048	1.18	0.76	1.10
		Line E	83	13.18	40.46	0.068	19.77	3.10	-0.056	1.17	0.79	1.09
		Line F	86	4.99	16.19	0.066	8.19	1.21	-0.058	1.19	0.77	1.10
	2nd	Line 3	89	9.30	32.53	0.075	15.11	2.32	-0.062	1.17	0.75	1.09
		Line 4	93	6.28	22.36	0.072	10.09	1.53	-0.055	1.16	0.75	1.09
		Line 5	97	8.25	28.17	0.070	12.97	2.00	-0.043	1.18	0.76	1.10
		Line B	100	13.52	46.82	0.073	21.68	3.35	-0.052	1.17	0.75	1.10
		Line C	104	1.72	6.21	0.073	2.81	0.43	-0.068	1.16	0.75	1.09
		Line E	107	8.75	29.55	0.071	13.76	2.16	-0.041	1.19	0.76	1.10

Table 4.12 shows the fundamental frequency computed based on the initial stiffness for each of the construction variants of the large-house index building retrofitted according to measure No. 2. The introduction OSB panels below and above all window and door openings causes only a slight increase in natural frequency (see Table 4.8). The exterior Stucco is the main contributor to the lateral stiffness of the building.

The three retrofitted variants of the large-house index building were then excited again along their short side by the Canoga Park record of the 1994 Northridge Earthquake scale to a Peak Ground Acceleration (PGA) of 0.50 g. Figure 4.12 presents the displacement time-histories at the roof level under this ground motion. Figure 4.13 presents the corresponding hysteresis loops of the OSB shear elements along wall line 5.

Table 4.12 Fundamental Frequencies of Retrofitted Large-Index Building.

Construction Variant	Fundamental Frequency (Hz)	Mode of Vibration
Poor-Quality	5.26	Y-direction
Typical-Quality	5.81	Y-direction
Superior-Quality	6.29	Y-direction

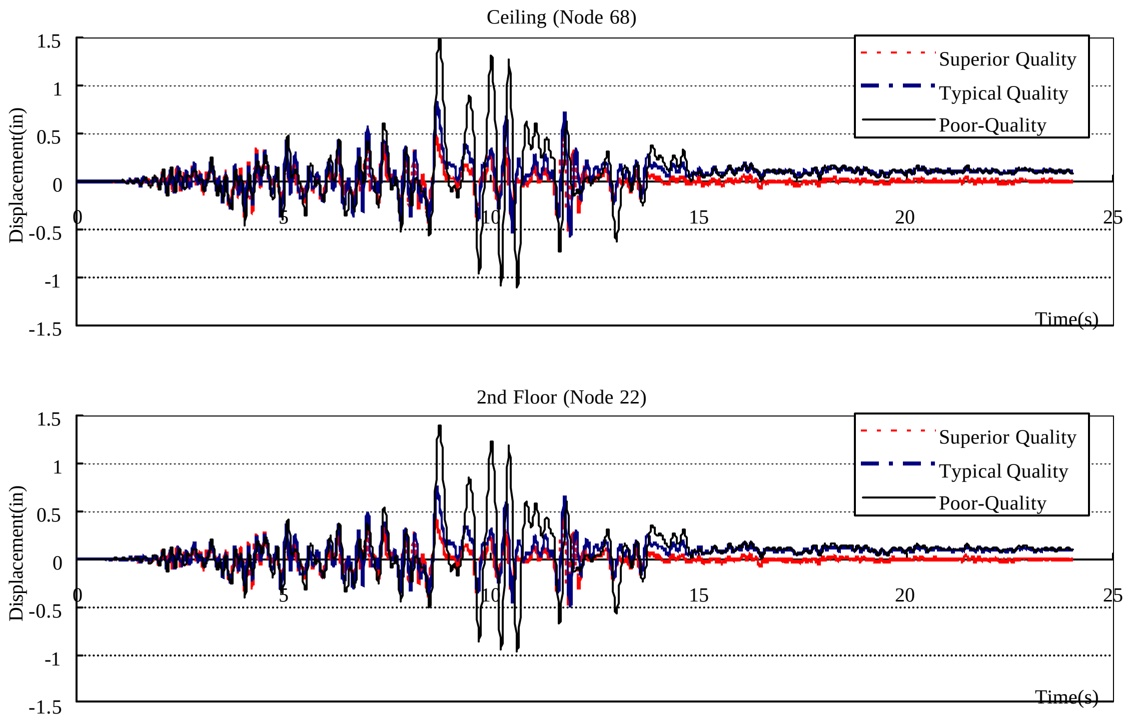


Figure 4.12 Displacement Time-Histories in the Y-Direction for Large-House Index Building Retrofitted According Measure No. 2 Under Canoga Park Record, PGA = 0.50 g.

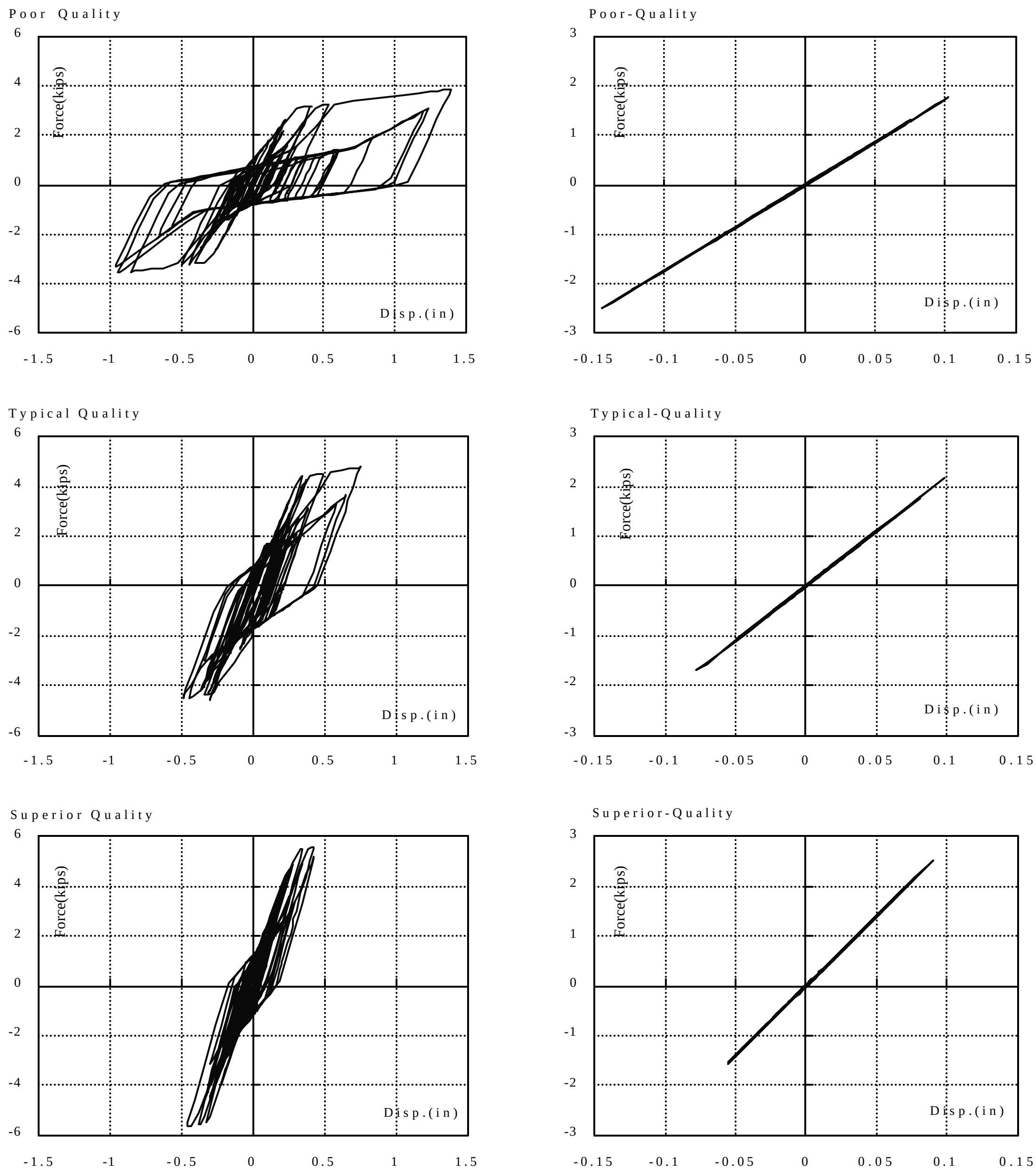


Figure 4.13 Hysteresis Loops of OSB Walls Along Wall Line 5 (Element 73 on the left side and 97 on the right side) for Large-House Index Building Retrofitted According to Measure No. 2 Under Canoga Park Record, PGA = 0.50 g.

4.12 Retrofit of Large House: Retrofit Measures No. 3 and No. 4

For the second retrofit measure of the large-house index buildings (Retrofit Measure No. 3), all shear walls were replaced by new OSB walls 15/ 32 in thick 10d common nail at an edge spacing of 3 inches. The hysteretic responses of 8-feet long segments of these retrofitted walls were computed by the CASHEW program, as shown in Table 4.13. The properties of each individual shear wall were then obtained by adjusting the values given in Table 4.13 for the actual length of the full wall piers in each wall line.

For the third retrofit measure of the large-house index buildings (Retrofit Measure No. 4), only the first floor wood shear walls were modified as shown in Figure 4.14. This alternate construction results from a code-level design using a rigid diaphragm distribution of forces for the first floor walls. The roof sheathing was treated as a flexible diaphragm. The CASHEW program also computed the hysteretic properties of these retrofitted first floor walls. Table 4.14 shows the resulting hysteretic parameters of these retrofitted first floor shear walls.

Both retrofit measures (3 and 4) were applied to the typical-quality variant of the large-house index building, which was excited again by the Canoga park record along its short side. The data files of the large-house index buildings retrofitted according to measures No. 3 and 4 are included in the CD-ROM accompanying this report.

Table 4.15 compares the fundamental frequencies and maximum responses obtained with these two retrofit measures with that of the original typical-quality construction variant of the large-house index building. Figure 4.15 shows the hysteretic response of the OSB shear walls along wall line 5 of the first floor of the large-house index building retrofitted according to measures No. 3 and 4. It can be observed that the retrofit measure No. 3 increased substantially the fundamental frequency of the original structure and enhanced significantly its seismic response. The seismic response of the structure retrofitted with measure No. 4, however, was almost identical to the response of the original structure.

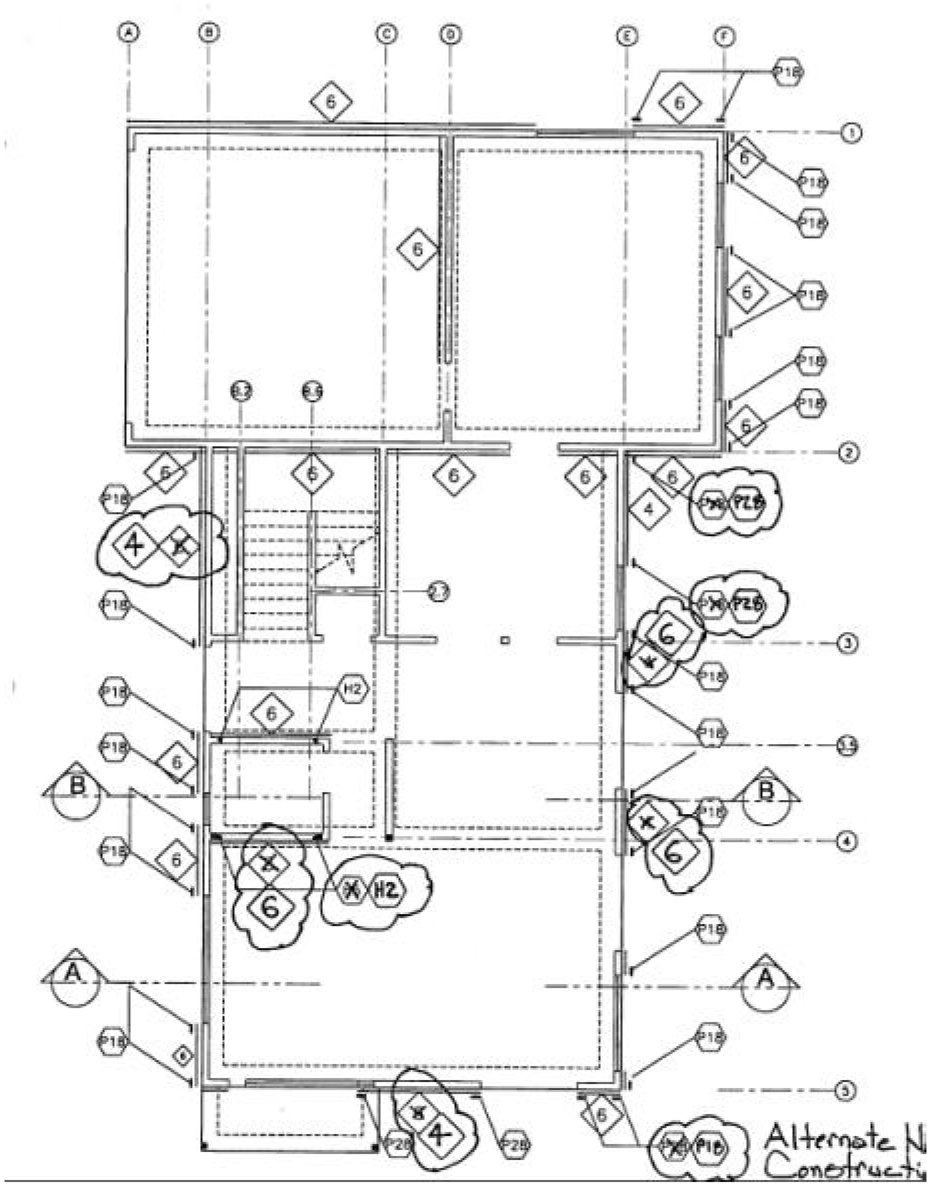


Figure 4.14 Modifications to First Floor OSB Shear Walls for Retrofit Measure No.4 of the Large-House Index Building.

Table 4.13 Parameters of Wayne Stewart Hysteresis Rule for 8-ft Segments of OSB Shear Walls of the Large-House Index Building Retrofitted with Measure No. 3.

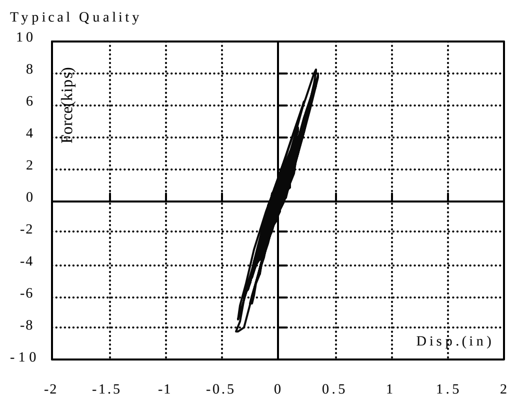
Wall Type	F_y (kips)	k_o (kips/in)	RF	F_u (kips)	F_i (kips)	PTRI	PUNL	a_1	b_1
OSB(8feet*8feet, thickness 15/32, edge nail spacing 3)	6.17	19.76	0.077	10.02	1.48	-0.082	1.07	0.76	1.09

Table 4.14 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Large-House Index Building Retrofitted with Measure No. 4.

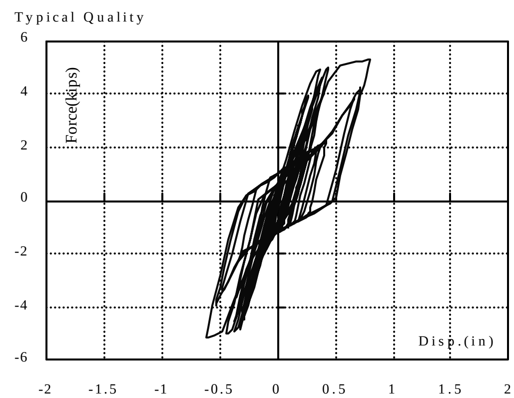
Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical shear wall OSB	1st	Line 4	70	2.91	8.73	0.064	4.69	0.71	-0.059	1.16	0.76	1.10
		Line 5	73	4.86	15.13	0.064	7.83	1.18	-0.059	1.16	0.76	1.10
		Line B	76	10.80	37.88	0.075	17.54	2.62	-0.071	1.13	0.75	1.09
		Line E	83	6.50	22.40	0.072	10.36	1.57	-0.055	1.14	0.76	1.09

Table 4.15 Fundamental Frequencies of Large-House Index Building Retrofitted According to Measures No. 3 and 4.

Construction Variant	Fundamental Frequency (Hz)	Maximum Response (in)	Direction of Vibration
Retrofit Measure No. 3	6.40	0.42	Y
Retrofit Measure No. 4	5.77	0.85	Y
Original Structure	5.80	0.84	Y



Retrofit Measured No. 3



Retrofit Measure No. 4

Figure 4.15 Hysteresis Loops of OSB Walls Along Wall Line 5 on the first floor (Element 73) for Large-House Index Building Retrofitted According to Measures No. 3 and 4 Under Canoga Park Record, PGA = 0.50 g.

5. MODELING OF INDEX BUILDING 3: TOWNHOUSE

5.1 General Description

The third index building considered in this study represents a two-story townhouse containing three units having each of approximately 1630 square feet of living space with an attached two-car garage. This building is assumed to have been built as a housing development "production house" in either the 1980's or 1990's, located in either Northern or Southern California. The design is based on engineered construction. Figure 5.1 shows plan views of the central unit. The architectural and structural drawings of the townhouse index building are included in Appendix C (Cobeen, 2001). The full building was modeled in the analysis, while only the end unit is being considered in the loss estimation study (Porter, 2001).

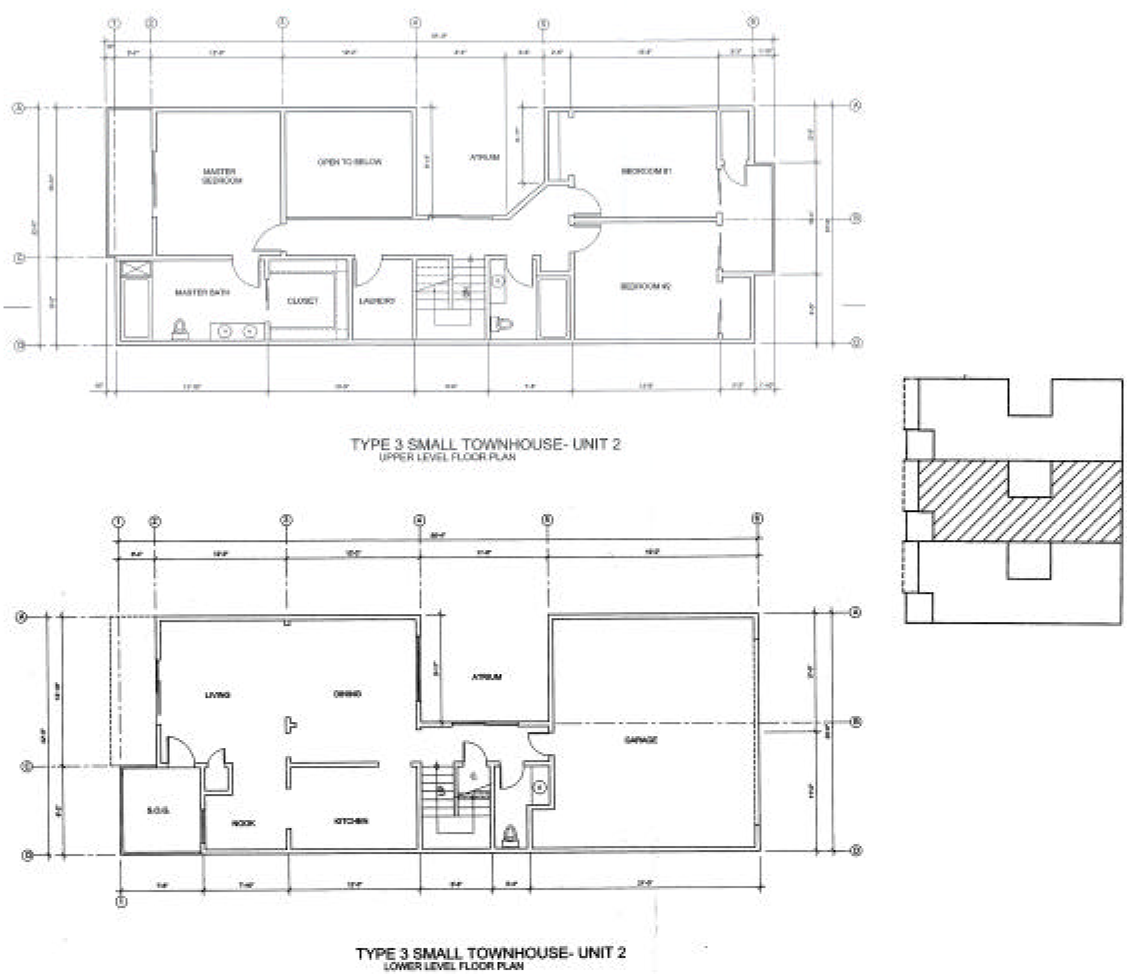


Figure 5.1 Plan Views of the Townhouse Index Building (Cobeen 2001).

The exterior walls of the townhouse index building are sheathed with stucco (UBC Tables 25B & 25C) on the outside along with wood shear walls (7/16 in thick OSB) and gypsum wallboards (1/2 in thick) on the inside. Furring nails (3/8-in head), spaced at 6 in on center along the vertical studs, are used to attach the wire mesh of the stucco finish to the wood framing. Eight-penny common nails spaced at 6, 4 or 3 in along the edges and 12 in on the field are used to attach the OSB panels to the framing. All interior gypsum walls are sheathed on both sides. Drywall nails (1-5/8 in long) spaced at 7 in on center along the vertical studs (spaced at 16-in on center) are used to attach all gypsum walls to the framing. The gypsum walls are assumed to be positioned vertically.

The floor diaphragm of the townhouse index building is composed of 2x12 joists spaced at 16 in on center and ¾" T&G PLWD or OSB sheathing. The townhouse index building is supported on a slab on grade. The roof diaphragm is built with 2x6 joists spaced at 24 in on center and ½" PLWD or 7/16" OSB sheathing. The ceiling is made of ½ in thick gypsum wallboard.

5.2 Description of Construction Variants

Three construction variants are defined for the townhouse index building. The three variants represent are poor-quality, typical-quality, and superior-quality construction, respectively. The characteristics of each construction variant are the same as of the large-house index building (see section 4.3).

5.3 Modeling Assumptions

The main assumptions used to develop the numerical pancake model of each construction variant of the townhouse index building are briefly discussed in this section.

Each stucco wall, OSB shear wall and gypsum wall is modeled by an in-plane shear element exhibiting the Wayne Stewart hysteresis rule, as described in Chapter 2. All exterior walls are sheathed with stucco and OSB on the outside and gypsum on the inside. Three parallel shear elements are used to simulate the in-plane behavior of the three sheathing materials. Interior walls consist of gypsum wallboards and some OSB shear walls. For each combined OSB and gypsum walls, two parallel shear elements are used as for the exterior walls. For each wall sheathed with gypsum on both sides, only one shear element per wall line is considered. The in-plane behavior of the floor diaphragm is modeled by linear-elastic quadrilateral finite elements. Adjacent nodes are connected together by in-plane frame elements having a very high axial stiffness.

5.4 Node Numbering

The pancake model of the townhouse index building incorporates three layers of nodes, as shown in Fig. 5.2.

The first floor nodes are located below the walls on the first story of the building. The earthquake ground motion is applied simultaneously at these nodes. The second floor nodes are located at the level of the second floor diaphragm and are used to connect the shear elements representing the walls on the first floor from the first floor level to the second floor diaphragm and to connect also the shear elements representing the vertical walls between the second floor diaphragm and the ceiling level. The ceiling nodes are used to connect the interior and exterior shear elements representing the vertical walls on the second floor. The nodes located at the common edges of two adjacent units are constraint to displace and rotate by the same amount. For example, node 29 is slaved to node 22.

5.5 Elements Description and Location

Eighty-two shear elements are used to represent the walls on the first floor of the townhouse index building. Figure 5.3 shows the location, orientation and number assigned to each of these shear elements in the RUAUMOKO data files. The numbers in the bracket following the element numbers represent the properties of the elements. Each unit is composed of the same properties except for the exterior stucco. The location, orientation and number assigned to the shear elements used to represent the walls on the second floor of the town-house index building are also illustrated in Figure 5.4.

The location, orientation and number assigned to the linear-elastic quadrilateral finite elements used to represent the floor and ceiling diaphragms of the townhouse index building are illustrated in Figures 5.5 and 5.6. The first figure represents the second floor diaphragm element in the second story portion and ceiling and roof diaphragm in the first story portion, while the second figure represents the ceiling and roof diaphragm element in the second story portion.

The location, orientation and number assigned to linear-elastic frame elements used along the edges of the floor and ceiling diaphragms to connect the corners of the quadrilateral finite elements to vertical wall elements are illustrated in Figures 5.7 and 5.8. The first figure represents the frame element, while the second figure represents the frame element in the second story portion.

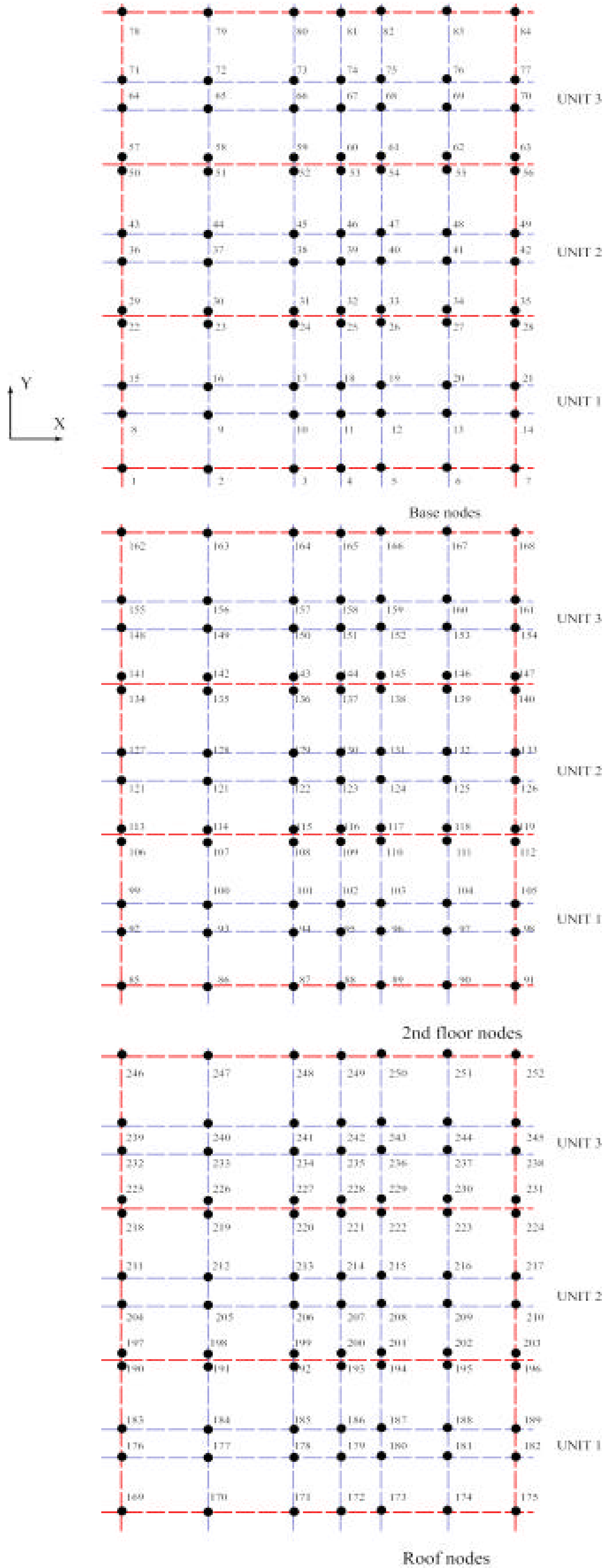


Figure 5.2 Node Numbering for Pancake Model of Townhouse Index Building.

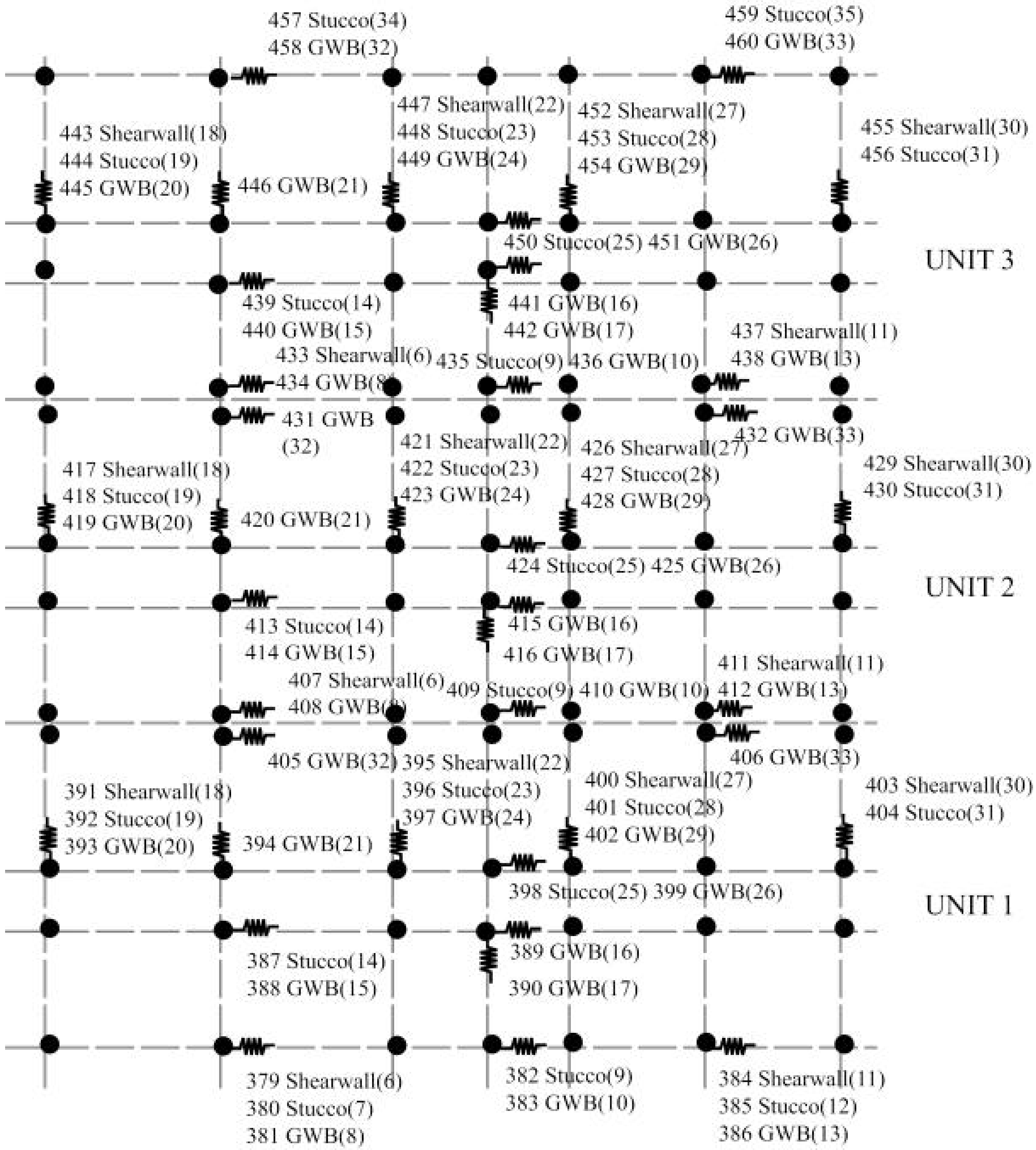


Figure 5.3 Locations, Orientation and Numbering of Shear Element for Interior and Exterior Vertical Walls on the First floor of the Townhouse Index Building.

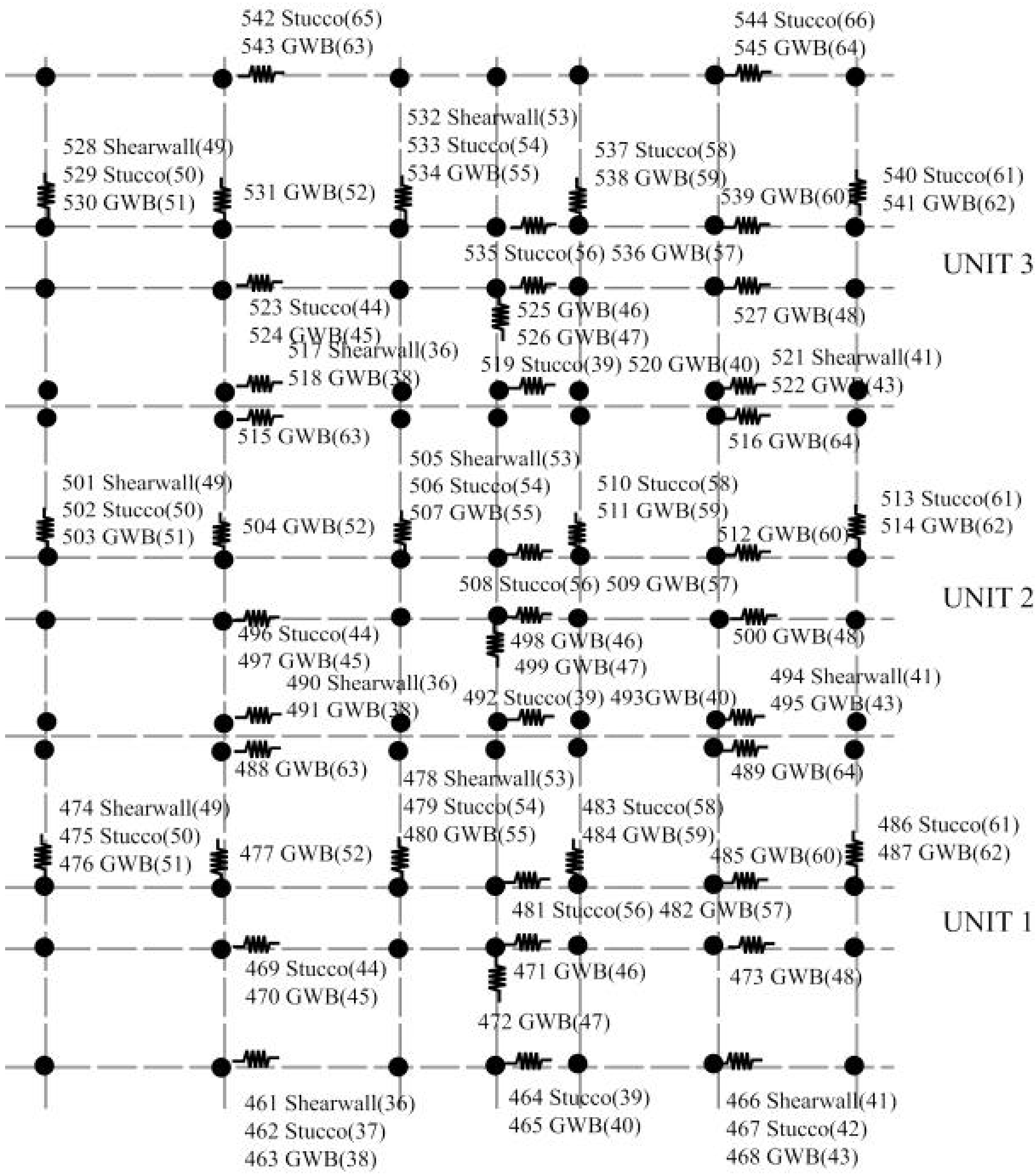


Figure 5.4 Locations, Orientation and Numbering of Shear Element for Interior and Exterior Vertical Walls on the Second Floor of the Townhouse Index Building.

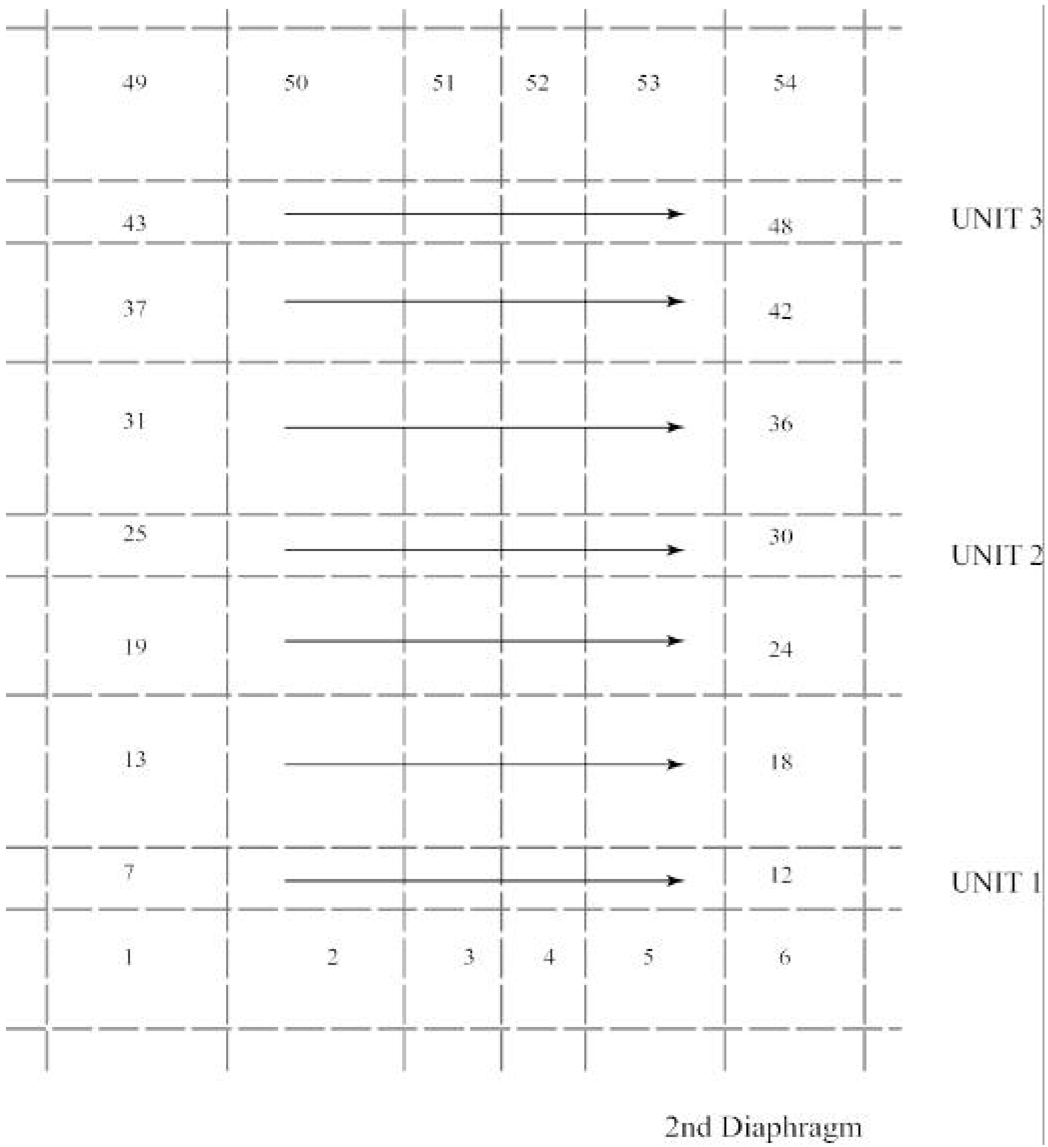


Figure 5.5 Locations, Orientation and Numbering of Quadrilateral Elements for the Second Floor Diaphragm of the Townhouse Index Building.

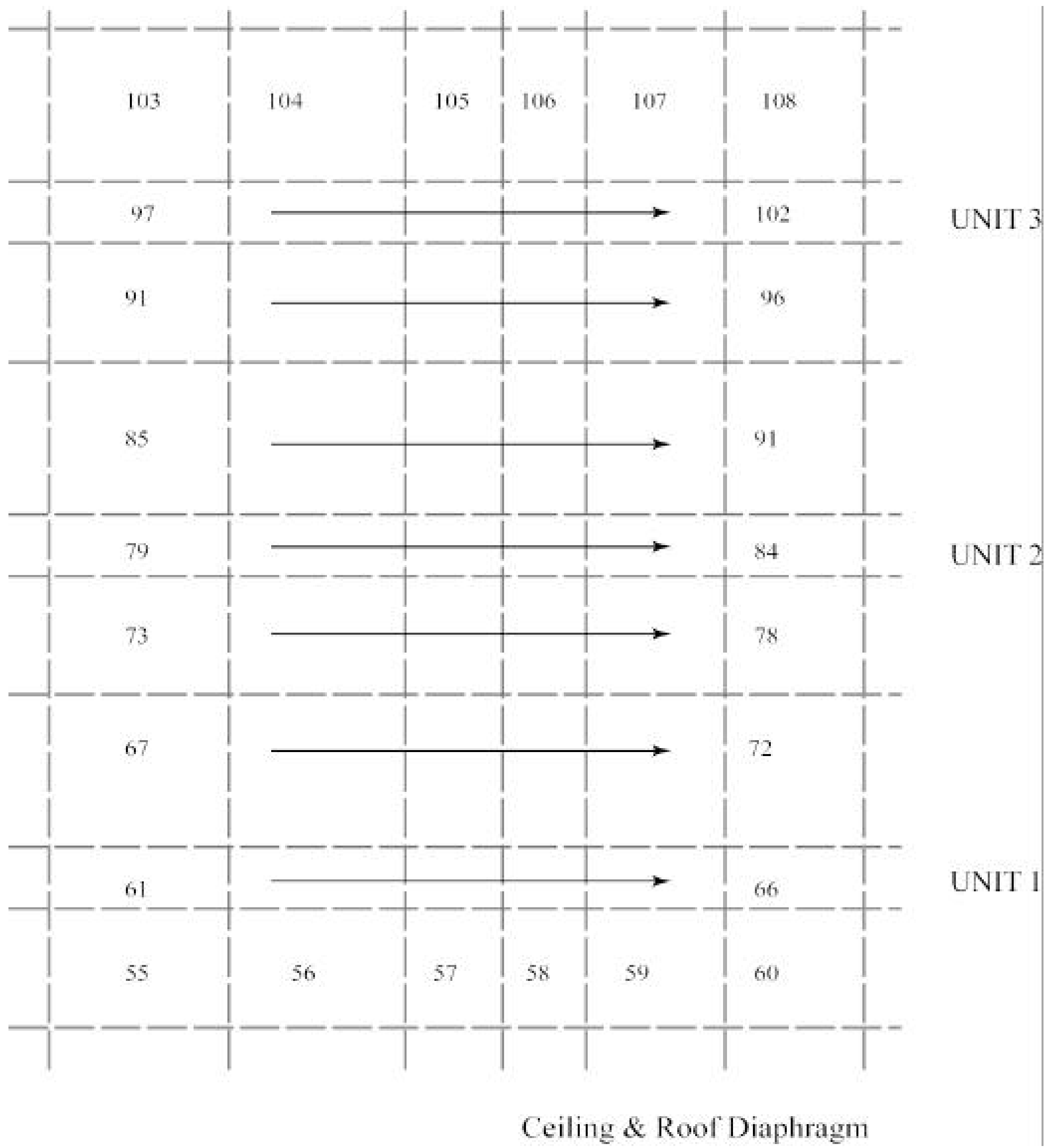


Figure 5.6 Locations, Orientation and Numbering of Quadrilateral Elements for the Roof Diaphragm of the Townhouse Index Building.

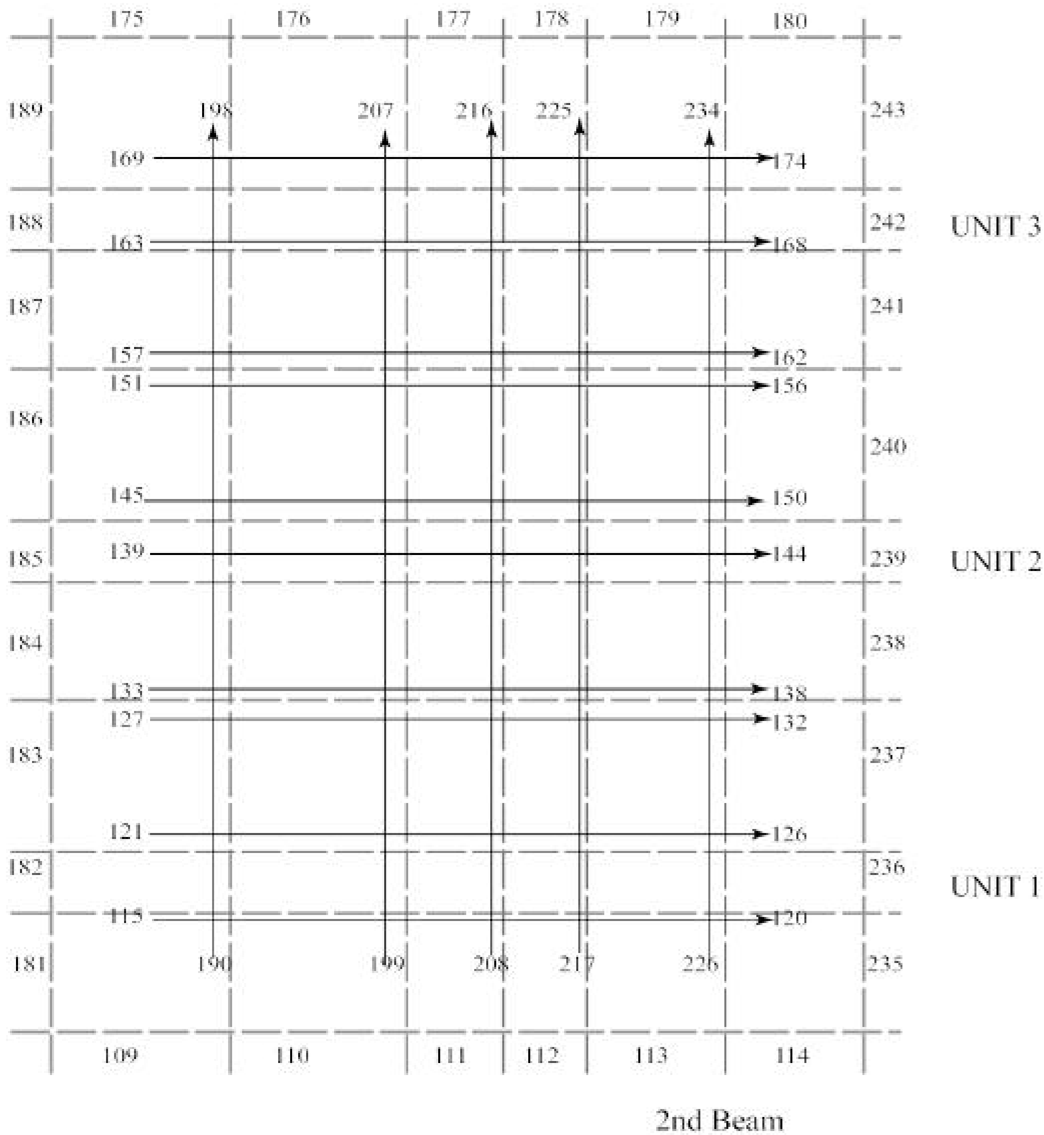


Figure 5.7 Locations, Orientation and Numbering of Frame Elements for the Second Floor of the Townhouse Index Building.

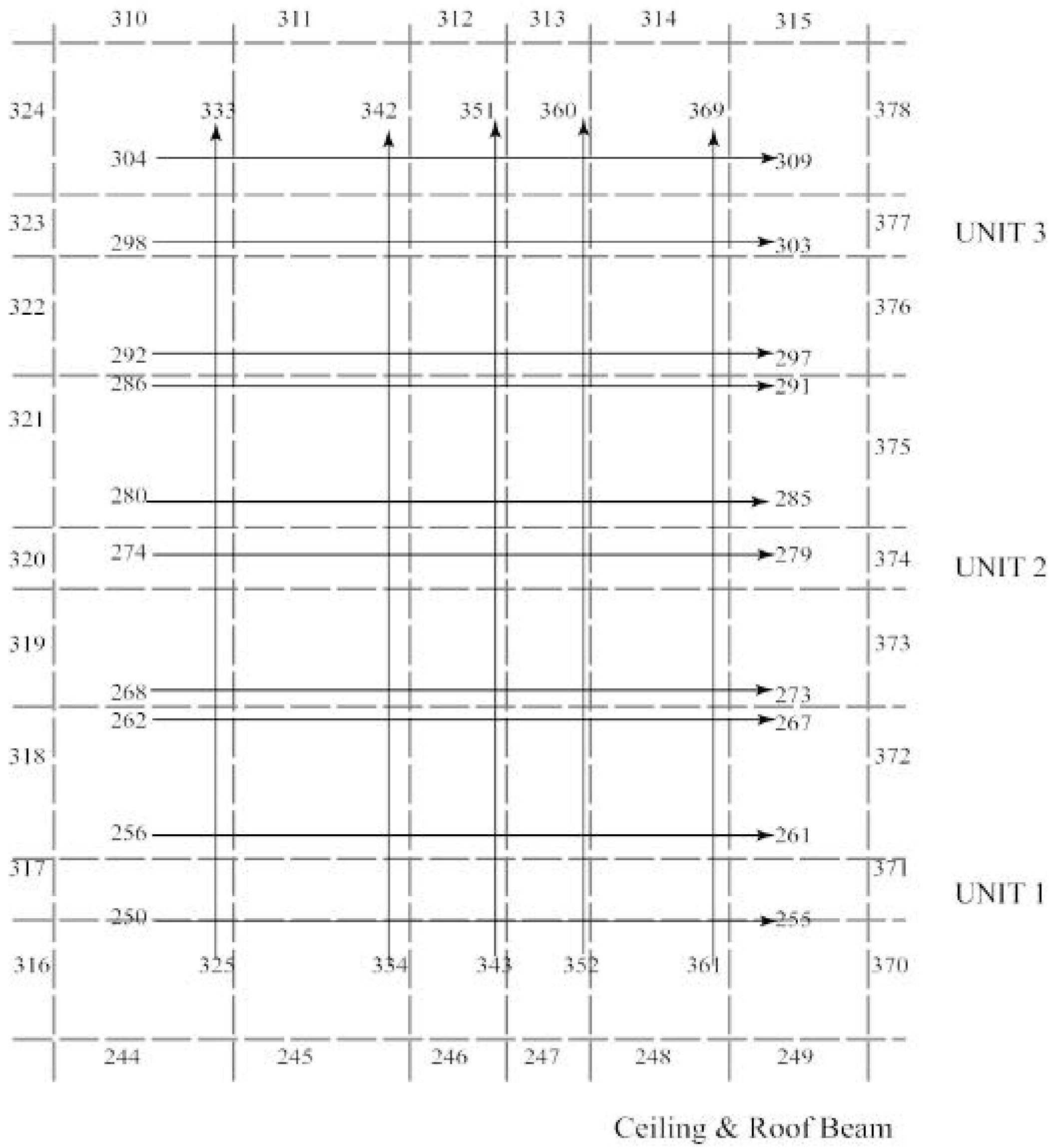


Figure 5.8 Locations, Orientation and Numbering of Frame Elements for the Roof of the Townhouse Index Building.

5.6 Hysteretic Parameters for Vertical Wall Shear Elements

The in-plane cyclic responses of the vertical walls incorporated in the townhouse index building were modeled by shear elements exhibiting the same Wayne Stewart Hysteresis rule. The hysteretic parameters for each stucco and gypsum wall were estimated from available cyclic test data on wall assemblies. The hysteretic parameters for the OSB shear wall were computed by the computer program CASHEW: Cyclic Analysis of wood SHEar Walls (Folz and Filiatrault, 2000) developed under Task 1.5.1 of the CUREE-Caltech Woodframe Project.

These tests of stucco and gypsum were conducted at the University of California at Irvine for the City of Los Angeles (COLA) Project (Pardoen, 2000). The parameters of the Wayne Stewart hysteresis of the stucco are the same as that of the small-house index building as shown in Table 3.2. The parameters for the gypsum walls are the same as that of the large- house index building. The hysteretic properties of the spring elements were obtained by adjusting the strength and stiffness values of Table 5.2 for the actual length of the full wall piers in each wall line. The same procedure was adopted for the interior gypsum sheathing of the exterior walls, but the values were divided by two since these walls are sheathed with gypsum wallboard only on one side.

The properties of the OSB shear walls were predicted by the computer program CASHEW: Cyclic Analysis of wood SHEar Walls (Folz and Filiatrault, 2000). The hysteretic parameters of the sheathing-to-framing connectors shown in Table 4.3 were used again for the townhouse index building. The nail schedule was assumed the same as that of the large-house index building. In the cyclic analysis of the poor and typical quality variants, there were some walls that were unable to be analyzed with the CASHEW program. For these cases, the backbone parameters such as Fu, Fy, FI, K_0 were obtained from the monotonic analysis results and the other parameters were assumed the same as the superior quality construction variant. The resulting properties are shown in Tables 5.1, 5.2, and 5.3 for the poor-quality, typical-quality and superior-quality variant of the townhouse index building, respectively.

Table 5.1 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Poor-Quality Variant of the Townhouse Index Building.

Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Shearwall OSB	1st	LineD 2-4	6	5.26	18.05	0.072	8.50	1.26	-0.066	1.16	0.75	1.09
		Line D 5-6	11	5.26	18.05	0.072	8.50	1.26	-0.066	1.16	0.75	1.09
		Line2	18	4.08	11.67	0.072	6.58	0.98	-0.065	1.15	0.76	1.09
		Line4	22	4.89	15.90	0.071	7.65	1.18	-0.046	1.14	0.77	1.10
		Line5	27	3.46	11.50	0.068	5.58	0.83	-0.036	1.19	0.77	1.10
		Line6	30	1.61	4.48	0.062	2.67	0.38	-0.064	1.18	0.78	1.10
	2nd	LineD 2-4	36	5.26	18.05	0.072	8.50	1.26	-0.066	1.16	0.75	1.09
		LineD 5-6	41	3.16	10.83	0.072	5.10	0.76	-0.066	1.16	0.75	1.09
		Line2	49	3.21	11.21	0.074	5.23	0.78	-0.074	1.12	0.75	1.09
		Line4	53	2.11	7.22	0.072	3.40	0.51	-0.066	1.16	0.75	1.09
Vertical Exterior Stucco	1st	LineD 2-4	7	3.37	38.70	0.082	5.05	0.85	-0.064	1.45	0.38	1.09
		LineD 4-5	9	1.98	22.77	0.082	2.97	0.50	-0.064	1.45	0.38	1.09
		LineD 5-6	12	3.26	37.40	0.082	4.88	0.82	-0.064	1.45	0.38	1.09
		LineC 2-4	14	0.32	3.58	0.082	0.47	0.08	-0.064	1.45	0.38	1.09
		LineB 4-5	25	0.94	10.74	0.082	1.40	0.24	-0.064	1.45	0.38	1.09
		LineA 2-4	34	2.75	31.55	0.082	4.12	0.69	-0.064	1.45	0.38	1.09
		LineA 5-6	35	3.26	37.40	0.082	4.88	0.82	-0.064	1.45	0.38	1.09
		Line2	19	2.21	25.37	0.082	3.31	0.56	-0.064	1.45	0.38	1.09
		Line4	23	0.67	7.65	0.082	1.00	0.17	-0.064	1.45	0.38	1.09
		Line5	28	1.69	19.35	0.082	2.53	0.43	-0.064	1.45	0.38	1.09
		Line6	31	0.85	9.76	0.082	1.28	0.22	-0.064	1.45	0.38	1.09
	2nd	LineD 2-4	37	4.64	53.33	0.082	6.96	1.16	-0.064	1.45	0.38	1.09
		LineD 4-5	39	1.98	22.77	0.082	2.97	0.50	-0.064	1.45	0.38	1.09
		LineD 5-6	42	3.26	37.40	0.082	4.88	0.82	-0.064	1.45	0.38	1.09
		LineC 2-4	44	0.57	6.51	0.082	0.85	0.15	-0.064	1.45	0.38	1.09
		LineB 4-5	56	0.94	10.74	0.082	1.40	0.24	-0.064	1.45	0.38	1.09
		LineA 2-4	65	2.41	27.64	0.082	3.61	0.61	-0.064	1.45	0.38	1.09
		LineA 5-6	66	2.58	29.59	0.082	3.86	0.65	-0.064	1.45	0.38	1.09
		Line2	50	2.72	31.22	0.082	4.08	0.68	-0.064	1.45	0.38	1.09
		Line4	54	1.69	19.35	0.082	2.53	0.43	-0.064	1.45	0.38	1.09
		Line5	58	1.69	19.35	0.082	2.53	0.43	-0.064	1.45	0.38	1.09
		Line6	61	2.01	23.09	0.082	3.02	0.51	-0.064	1.45	0.38	1.09
Vertical Gypsum	1st	LineD 2-4	8	2.35	35.33	0.064	3.19	0.59	-0.042	1.45	0.38	1.09
		LineD 4-5	10	1.39	20.79	0.064	1.88	0.35	-0.042	1.45	0.38	1.09
		LineD 5-6	13	2.27	34.15	0.064	3.08	0.57	-0.042	1.45	0.38	1.09
		LineC 2-4	15	2.35	35.33	0.064	3.19	0.59	-0.042	1.45	0.38	1.09
		LineC 4-5	16	1.54	23.16	0.064	2.09	0.39	-0.042	1.45	0.38	1.09
		LineB 4-5	26	0.66	9.80	0.064	0.89	0.17	-0.042	1.45	0.38	1.09
		LineA 2-4	32	1.92	28.80	0.064	2.60	0.48	-0.042	1.45	0.38	1.09
		LineA 4-5	33	2.27	34.15	0.064	3.08	0.57	-0.042	1.45	0.38	1.09
		Line2	20	1.90	28.50	0.064	2.57	0.48	-0.042	1.45	0.38	1.09
		Line3	21	1.09	16.33	0.064	1.48	0.28	-0.042	1.45	0.38	1.09
		Line4	24	2.41	36.22	0.064	3.27	0.61	-0.042	1.45	0.38	1.09
		Line45	17	1.94	29.10	0.064	2.63	0.49	-0.042	1.45	0.38	1.09
		Line5	29	3.32	49.88	0.064	4.50	0.83	-0.042	1.45	0.38	1.09
	2nd	LineD 2-4	38	3.24	48.69	0.064	4.39	0.81	-0.042	1.45	0.38	1.09
		LineD 4-5	40	1.39	20.79	0.064	1.88	0.35	-0.042	1.45	0.38	1.09
		LineD 5-6	43	2.27	34.15	0.064	3.08	0.57	-0.042	1.45	0.38	1.09
		LineC 2-4	45	5.01	75.41	0.064	6.80	1.26	-0.042	1.45	0.38	1.09
		LineC 4-5	46	1.50	22.57	0.064	2.04	0.38	-0.042	1.45	0.38	1.09
		LineC 5-6	48	1.35	20.19	0.064	1.82	0.34	-0.042	1.45	0.38	1.09
		LineB 4-5	57	0.66	9.80	0.064	0.89	0.17	-0.042	1.45	0.38	1.09
		Line B 5-6	60	3.57	53.74	0.064	4.85	0.90	-0.042	1.45	0.38	1.09
		LineA 2-4	63	1.68	25.24	0.064	2.28	0.42	-0.042	1.45	0.38	1.09
		LineA 5-6	64	1.80	27.02	0.064	2.44	0.45	-0.042	1.45	0.38	1.09
		Line2	51	1.90	28.50	0.064	2.57	0.48	-0.042	1.45	0.38	1.09
		Line3	52	4.68	70.36	0.064	6.35	1.17	-0.042	1.45	0.38	1.09
		Line4	55	4.08	61.31	0.064	5.53	1.02	-0.042	1.45	0.38	1.09
		Line45	47	1.94	29.10	0.064	2.63	0.49	-0.042	1.45	0.38	1.09
		Line5	59	3.11	46.76	0.064	4.22	0.78	-0.042	1.45	0.38	1.09
		Line 6	62	3.18	47.80	0.064	4.31	0.80	-0.042	1.45	0.38	1.09

Table 5.2 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Typical-Quality Variant of the Townhouse Index Building.

Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Shearwall OSB	1st	LineD 2-4	6	7.22	26.73	0.072	11.79	1.77	-0.066	1.16	0.75	1.09
		Line D 5-6	11	7.22	26.73	0.072	11.79	1.77	-0.066	1.16	0.75	1.09
		Line2	18	5.31	14.62	0.072	8.57	1.29	-0.065	1.15	0.76	1.09
		Line4	22	6.55	20.69	0.072	10.19	1.57	-0.045	1.12	0.78	1.10
		Line5	27	5.00	16.82	0.068	7.72	1.21	-0.036	1.19	0.77	1.10
		Line6	30	2.13	5.81	0.063	3.53	0.51	-0.063	1.16	0.78	1.10
	2nd	LineD 2-4	36	7.22	26.73	0.072	11.79	1.77	-0.066	1.16	0.75	1.09
		LineD 5-6	41	4.33	16.03	0.072	7.08	1.06	-0.066	1.16	0.75	1.09
		Line2	49	4.31	14.61	0.076	7.02	1.05	-0.074	1.10	0.76	1.09
		Line4	53	2.89	10.70	0.072	4.72	0.71	-0.066	1.16	0.75	1.09
Vertical Exterior Stucco	1st	LineD 2-4	7	4.33	49.76	0.082	6.49	1.09	-0.064	1.45	0.38	1.09
		LineD 4-5	9	2.55	29.27	0.082	3.82	0.64	-0.064	1.45	0.38	1.09
		LineD 5-6	12	4.19	48.08	0.082	6.28	1.05	-0.064	1.45	0.38	1.09
		LineC 2-4	14	0.40	4.60	0.082	0.60	0.10	-0.064	1.45	0.38	1.09
		LineB 4-5	25	1.20	13.80	0.082	1.80	0.30	-0.064	1.45	0.38	1.09
		LineA 2-4	34	3.53	40.56	0.082	5.29	0.89	-0.064	1.45	0.38	1.09
		LineA 5-6	35	4.19	48.08	0.082	6.28	1.05	-0.064	1.45	0.38	1.09
		Line2	19	2.84	32.61	0.082	4.26	0.71	-0.064	1.45	0.38	1.09
		Line4	23	0.86	9.83	0.082	1.29	0.22	-0.064	1.45	0.38	1.09
		Line5	28	2.17	24.88	0.082	3.25	0.55	-0.064	1.45	0.38	1.09
		Line6	31	1.10	12.55	0.082	1.64	0.28	-0.064	1.45	0.38	1.09
	2nd	LineD 2-4	37	5.97	68.57	0.082	8.95	1.50	-0.064	1.45	0.38	1.09
		LineD 4-5	39	2.55	29.27	0.082	3.82	0.64	-0.064	1.45	0.38	1.09
		LineD 5-6	42	4.19	48.08	0.082	6.28	1.05	-0.064	1.45	0.38	1.09
		LineC 2-4	44	0.73	8.37	0.082	1.10	0.19	-0.064	1.45	0.38	1.09
		LineB 4-5	56	1.20	13.80	0.082	1.80	0.30	-0.064	1.45	0.38	1.09
		LineA 2-4	65	3.09	35.54	0.082	4.64	0.78	-0.064	1.45	0.38	1.09
		LineA 5-6	66	3.31	38.05	0.082	4.97	0.83	-0.064	1.45	0.38	1.09
		Line2	50	3.49	40.14	0.082	5.24	0.88	-0.064	1.45	0.38	1.09
		Line4	54	2.17	24.88	0.082	3.25	0.55	-0.064	1.45	0.38	1.09
		Line5	58	2.17	24.88	0.082	3.25	0.55	-0.064	1.45	0.38	1.09
		Line6	61	2.59	29.69	0.082	3.88	0.65	-0.064	1.45	0.38	1.09
Vertical Gypsum	1st	LineD 2-4	8	2.66	40.04	0.064	3.61	0.67	-0.042	1.45	0.38	1.09
		LineD 4-5	10	1.57	23.56	0.064	2.13	0.40	-0.042	1.45	0.38	1.09
		LineD 5-6	13	2.58	38.70	0.064	3.49	0.65	-0.042	1.45	0.38	1.09
		LineC 2-4	15	2.66	40.04	0.064	3.61	0.67	-0.042	1.45	0.38	1.09
		LineC 4-5	16	1.75	26.25	0.064	2.37	0.44	-0.042	1.45	0.38	1.09
		LineB 4-5	26	0.74	11.11	0.064	1.01	0.19	-0.042	1.45	0.38	1.09
		LineA 2-4	32	2.17	32.64	0.064	2.95	0.55	-0.042	1.45	0.38	1.09
		LineA 4-5	33	2.58	38.70	0.064	3.49	0.65	-0.042	1.45	0.38	1.09
		Line2	20	2.15	32.30	0.064	2.92	0.54	-0.042	1.45	0.38	1.09
		Line3	21	1.23	18.51	0.064	1.67	0.31	-0.042	1.45	0.38	1.09
		Line4	24	2.73	41.05	0.064	3.70	0.69	-0.042	1.45	0.38	1.09
		Line45	17	2.20	32.98	0.064	2.98	0.55	-0.042	1.45	0.38	1.09
		Line5	29	3.76	56.53	0.064	5.10	0.94	-0.042	1.45	0.38	1.09
	2nd	LineD 2-4	38	3.67	55.18	0.064	4.98	0.92	-0.042	1.45	0.38	1.09
		LineD 4-5	40	1.57	23.56	0.064	2.13	0.40	-0.042	1.45	0.38	1.09
		LineD 5-6	43	2.58	38.70	0.064	3.49	0.65	-0.042	1.45	0.38	1.09
		LineC 2-4	45	5.68	85.47	0.064	7.71	1.42	-0.042	1.45	0.38	1.09
		LineC 4-5	46	1.70	25.58	0.064	2.31	0.43	-0.042	1.45	0.38	1.09
		LineC 5-6	48	1.52	22.88	0.064	2.07	0.38	-0.042	1.45	0.38	1.09
		LineB 4-5	57	0.74	11.11	0.064	1.01	0.19	-0.042	1.45	0.38	1.09
		Line B 5-6	60	4.05	60.90	0.064	5.49	1.02	-0.042	1.45	0.38	1.09
		LineA 2-4	63	1.90	28.60	0.064	2.58	0.48	-0.042	1.45	0.38	1.09
		LineA 5-6	64	2.04	30.62	0.064	2.76	0.51	-0.042	1.45	0.38	1.09
		Line2	51	2.15	32.30	0.064	2.92	0.54	-0.042	1.45	0.38	1.09
		Line3	52	5.30	79.75	0.064	7.19	1.33	-0.042	1.45	0.38	1.09
		Line4	55	4.62	69.48	0.064	6.27	1.16	-0.042	1.45	0.38	1.09
		Line45	47	2.20	32.98	0.064	2.98	0.55	-0.042	1.45	0.38	1.09
		Line5	59	3.52	53.00	0.064	4.78	0.88	-0.042	1.45	0.38	1.09
		Line 6	62	3.60	54.17	0.064	4.89	0.90	-0.042	1.45	0.38	1.09

Table 5.3 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Superior-Quality Variant of the Townhouse Index Building.

Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Shearwall OSB	1st	LineD 2-4	6	8.58	31.06	0.073	14.01	2.11	-0.068	1.16	0.75	1.09
		Line D 5-6	11	8.58	31.06	0.073	14.01	2.11	-0.068	1.16	0.75	1.09
		Line2	18	6.36	17.02	0.072	10.32	1.55	-0.065	1.15	0.76	1.09
		Line4	22	7.17	22.46	0.072	11.63	1.74	-0.065	1.15	0.76	1.09
		Line5	27	5.94	19.61	0.069	9.16	1.43	-0.037	1.18	0.77	1.10
		Line6	30	2.57	6.90	0.064	4.26	0.61	-0.065	1.15	0.78	1.10
	2nd	LineD 2-4	36	8.58	31.06	0.073	14.01	2.11	-0.068	1.16	0.75	1.09
		LineD 5-6	41	5.15	18.64	0.073	8.41	1.27	-0.068	1.16	0.75	1.09
		Line2	49	4.89	16.36	0.076	7.96	1.19	-0.078	1.10	0.76	1.09
		Line4	53	3.44	12.42	0.073	5.61	0.85	-0.068	1.16	0.75	1.09
Vertical Exterior Stucco	1st	LineD 2-4	7	4.81	55.28	0.082	7.21	1.21	-0.064	1.45	0.38	1.09
		LineD 4-5	9	2.83	32.52	0.082	4.25	0.71	-0.064	1.45	0.38	1.09
		LineD 5-6	12	4.65	53.42	0.082	6.97	1.17	-0.064	1.45	0.38	1.09
		LineC 2-4	14	0.45	5.11	0.082	0.67	0.12	-0.064	1.45	0.38	1.09
		LineB 4-5	25	1.34	15.33	0.082	2.00	0.34	-0.064	1.45	0.38	1.09
		LineA 2-4	34	3.92	45.06	0.082	5.88	0.98	-0.064	1.45	0.38	1.09
		LineA 5-6	35	4.65	53.42	0.082	6.97	1.17	-0.064	1.45	0.38	1.09
		Line2	19	3.16	36.24	0.082	4.73	0.79	-0.064	1.45	0.38	1.09
		Line4	23	0.95	10.92	0.082	1.43	0.24	-0.064	1.45	0.38	1.09
		Line5	28	2.41	27.64	0.082	3.61	0.61	-0.064	1.45	0.38	1.09
		Line6	31	1.22	13.94	0.082	1.82	0.31	-0.064	1.45	0.38	1.09
	2nd	LineD 2-4	37	6.63	76.19	0.082	9.94	1.66	-0.064	1.45	0.38	1.09
		LineD 4-5	39	2.83	32.52	0.082	4.25	0.71	-0.064	1.45	0.38	1.09
		LineD 5-6	42	4.65	53.42	0.082	6.97	1.17	-0.064	1.45	0.38	1.09
		LineC 2-4	44	0.81	9.30	0.082	1.22	0.21	-0.064	1.45	0.38	1.09
		LineB 4-5	56	1.34	15.33	0.082	2.00	0.34	-0.064	1.45	0.38	1.09
		LineA 2-4	65	3.44	39.49	0.082	5.15	0.86	-0.064	1.45	0.38	1.09
		LineA 5-6	66	3.68	42.28	0.082	5.52	0.92	-0.064	1.45	0.38	1.09
		Line2	50	3.88	44.60	0.082	5.82	0.97	-0.064	1.45	0.38	1.09
		Line4	54	2.41	27.64	0.082	3.61	0.61	-0.064	1.45	0.38	1.09
		Line5	58	2.41	27.64	0.082	3.61	0.61	-0.064	1.45	0.38	1.09
		Line6	61	2.87	32.99	0.082	4.31	0.72	-0.064	1.45	0.38	1.09
Vertical Gypsum	1st	LineD 2-4	8	3.13	47.11	0.064	4.25	0.79	-0.042	1.45	0.38	1.09
		LineD 4-5	10	1.85	27.71	0.064	2.50	0.47	-0.042	1.45	0.38	1.09
		LineD 5-6	13	3.03	45.53	0.064	4.11	0.76	-0.042	1.45	0.38	1.09
		LineC 2-4	15	3.13	47.11	0.064	4.25	0.79	-0.042	1.45	0.38	1.09
		LineC 4-5	16	2.06	30.88	0.064	2.79	0.52	-0.042	1.45	0.38	1.09
		LineB 4-5	26	0.87	13.07	0.064	1.18	0.22	-0.042	1.45	0.38	1.09
		LineA 2-4	32	2.56	38.40	0.064	3.47	0.64	-0.042	1.45	0.38	1.09
		LineA 4-5	33	3.03	45.53	0.064	4.11	0.76	-0.042	1.45	0.38	1.09
		Line2	20	2.53	38.00	0.064	3.43	0.64	-0.042	1.45	0.38	1.09
		Line3	21	1.45	21.78	0.064	1.97	0.37	-0.042	1.45	0.38	1.09
		Line4	24	3.21	48.30	0.064	4.36	0.81	-0.042	1.45	0.38	1.09
		Line45	17	2.58	38.80	0.064	3.50	0.65	-0.042	1.45	0.38	1.09
		Line5	29	4.42	66.50	0.064	6.00	1.11	-0.042	1.45	0.38	1.09
	2nd	LineD 2-4	38	4.32	64.92	0.064	5.86	1.08	-0.042	1.45	0.38	1.09
		LineD 4-5	40	1.85	27.71	0.064	2.50	0.47	-0.042	1.45	0.38	1.09
		LineD 5-6	43	3.03	45.53	0.064	4.11	0.76	-0.042	1.45	0.38	1.09
		LineC 2-4	45	6.68	100.55	0.064	9.07	1.67	-0.042	1.45	0.38	1.09
		LineC 4-5	46	2.00	30.09	0.064	2.72	0.50	-0.042	1.45	0.38	1.09
		LineC 5-6	48	1.79	26.92	0.064	2.43	0.45	-0.042	1.45	0.38	1.09
		LineB 4-5	57	0.87	13.07	0.064	1.18	0.22	-0.042	1.45	0.38	1.09
		Line B 5-6	60	4.76	71.65	0.064	6.46	1.19	-0.042	1.45	0.38	1.09
		LineA 2-4	63	2.24	33.65	0.064	3.04	0.56	-0.042	1.45	0.38	1.09
		LineA 5-6	64	2.40	36.03	0.064	3.25	0.60	-0.042	1.45	0.38	1.09
		Line2	51	2.53	38.00	0.064	3.43	0.64	-0.042	1.45	0.38	1.09
		Line3	52	6.24	93.82	0.064	8.46	1.56	-0.042	1.45	0.38	1.09
		Line4	55	5.43	81.74	0.064	7.37	1.36	-0.042	1.45	0.38	1.09
		Line45	47	2.58	38.80	0.064	3.50	0.65	-0.042	1.45	0.38	1.09
		Line5	59	4.15	62.35	0.064	5.62	1.04	-0.042	1.45	0.38	1.09
		Line 6	62	4.24	63.73	0.064	5.75	1.06	-0.042	1.45	0.38	1.09

5.7 Properties of Horizontal Floor and Roof Diaphragms

The in-plane stiffness of the floor diaphragm, G_d, is taken to be 800 kips/in, while the corresponding value for the roof diaphragm is take to be 400 kips/in. These values are prescribed by the NEHRP Guidelines for Seismic Rehabilitation of Buildings (FEMA, 1997). Each linear-elastic diaphragm finite element is assigned elastic properties such that:

$$Gt = \frac{Et}{2(1+\nu)} = G_d$$

where G is the equivalent shear modulus, E is the equivalent elastic modulus, ν is the equivalent Poisson's ratio and t is the thickness of the finite element. The in-plane stiffness of the atrium portion is take to be zero.

5.8 Properties of frame elements

The frame elements used to connect the shearwalls to the floor and roof diaphragms are assumed as rigid truss elements. In the atrium portion the townhouse index building, these elements are assumed zero stiffness.

5.9 Weight Distribution

Table 5.4 list the weights considered for the townhouse index building. These weights were distributed as nodal lumped seismic weights according to the tributary areas of the nodes (see Figure 5.2).

Table 5.4 Weights for Town-House Index Building.

Item	Location		Length (ft)	Width or Height (ft)	Area (sq ft)	Unit Weight (psf)	Weight #	Total Weight #
Roof			60	22	1320.0	14	16,945	16,945
		Less Atrium			109.7			
		net			1210.4			
Ceiling			58.33	22	1283.3	3.5	4,108	4,108
		Less Atrium			109.7			
		net			1173.6			
		Stucco Sofit	13.83	4.17	57.7	6	346	346
			10	3	30.0	6	180	180
			8.17	0.82	6.7	6	40	40
						Total Roof & Ceiling		**21,619**
2 nd Flr Walls	Line A 2-4 party	Total	24.17	8	193.4	3.7	715	715
	Line A 2-4	Attic dw	28.33	5	141.7	2	283	283
	Line A 5-6 party	Total	19.17	8	153.4	3.7	567	567
	Line A 5-6	Attic dw	19.17	5	95.9	2	192	192
	Line B-rail 3-4	Total	12.17	3	36.5	5.3	194	194
	Line B-Atr 4-5	Total	13	16	208.0			2,430
		open & door	6	3.33	20.0	4	80	
		typ ext			188.0	12.5	2,350	
	Line B 1-2 int	Total	13.17	8	105.4	5.3	558	558
	Line C 1-2 Ext	Total	3.33	8	26.6	12.5	333	333
	Line C- 2-4.3	Total	27.5	8	220.0			1,056
		window/door	5	6.67	33.4	2	67	
		typ int			186.7	5.3	989	
	Line C 4.5-5.1	Total	7.75	8	62.0			273
		door	2.5	6.7	16.8	2	34	
		typ int			45.3	5.3	240	
	Line D 1-6 Party	Total	58.33	8	466.6	3.7	1,727	1,727
	Line D attic	total	58.33	5	291.7	2	583	583

Line 1 C-D ext	Total	8.17	8	65.4	12.5	817	817
Line 2 C -A	Total	13.82	8	110.6			1,042
	open & door	6	6.67	40.0	4	160	
	typ ext			70.5	12.5	882	
Line 2.9 D-C	Total	8.17	8	65.4			236
	open & door	5	6.7	33.5	2	67	
	typ interior			31.9	5.3	169	
Line 3 C-A	Total	13.67	8	109.4			525
	open & door	2.5	6.67	16.7	2	33	
	typ interior			92.7	5.3	491	
Line 3.5 D-C int.	Total	8.17	8	65.4	5.3	346	346
Line 4 D-C int	Total	8.17	8	65.4	5.3	346	346
Line 4 B-A ext	Total	9.9	18	178.2	12.5	2,228	2,228
Line 4.5 D-C int.	Total	8.17	8	65.4	5.3	346	346
Line 4.9 C.1-C ext	Total	2	8	16.0	12.5	200	200
Line 5.0- A-A.7ext	Total	7	15	105.0	12.5	1,313	1,313
Line 5.1 D - B	Total	11	8	88.0			411
	window/door	2.5	6.7	16.8	2	34	
	typ int			71.3	5.3	378	
Line 5.1 B - A	Total	11	8	88.0			301
	window/door	7.5	6.7	50.3	2	101	
	typ int			37.8	5.3	200	
Line 5.9- A-A.5	Total	6	8	48.0	5.3	254	254
Line 5.9 A.5 B.5	Total	10.33	8	82.6			520
	window/door	9	6.7	60.3	4	241	
	typ ext			22.3	12.5	279	
Line 5.9 B.5-D	Total	6	8	48.0			144
	window/door	5	6.7	33.5	2	67	
	typ int			14.5	5.3	77	
Line 6 d-a	Total	22	8	176.0			1,299
	openings	10.3	7	72.1	0	0	
	typ ext			103.9	12.5	1,299	

						Total 2nd floor walls		**19,240**
2nd Flr			58.33	22	1283.3	9	10836	10,836
		less atrium			-109.6			
		+ balcony	13.82	0.83	11.5			
			10.33	1.83	18.9			
		net			1204.0			
		stucco sofit	13.83	4.17	57.7	6	346	346
			10	5	50.0	6	300	300
	atrium triangle	one half	3.5	3.5	6.1	6	37	37
						Total 2nd floor		**11,519**
	Line A 2-4	party	24.17	8	193.4	3.7	715	715
First Flr Walls	Line A 5-6	party@gar	19.17	8	153.4	4.1	629	629
	Line B 4-5	ext	11.67	8	93.4			827
		window/door	6	6.67	40.0	4	160	
		typ ext			53.3	12.5	667	
	Line C 2-2.2	ext	4	8	32.0			216
		window/door	2.5	7	17.5	2	35	
		typ ext			14.5	12.5	181	
	Line C 2.2-2.5	int	3	8	24.0			83
		window/door	2	6.7	13.4	2	27	
		typ int			10.6	5.3	56	
	Line C 3 - 4	total	12.2	8	97.6			393
		opening	3.5	6.7	23.5	0	0	
		typ int.			74.2	5.3	393	
	Line C 4.3-5	int	8	8	64.0			229
		window/door	5	6.7	33.5	2	67	
		typ int			30.5	5.3	162	
	Line C.3 2.2-2.5	int	3	8	24.0	5.3	127	127
	Line C.5 4.7-5	int	2	8	16.0	5.3	85	85
	Line D 2.2-4.7	party	30	8	240.0	3.7	888	888
	Line D 4.7 - 6	party@gar	21	8	168.0	4.1	689	689
	Line 2 C-A	Total	12.83	8	102.6			926

	window/door	6	7	42.0	4	168	
	typ ext			60.6	12.5	758	
Line 2.2 D-C	Total	8.17	8	65.4			701
	window/door	4	3.4	13.6	4	54	
	typ ext			51.8	12.5	647	
Line 2.5 C-C.3	Int.	3	8	24.0	5.3	127	127
Line 3 A-D	Total	22	8	176.0			413
	openings	14	7	98.0	0	0	
	typ int			78.0	5.3	413	
Line 4 D-C int	Total	8.17	8	65.4	5.3	346	346
Line 4 B-A ext	Total	9.92	8	79.4			819
	window/door	6	3.4	20.4	4	82	
	typ ext			59.0	12.5	737	
Line 4.5 C-D int	Total	8.17	8	65.4	5.3	65	346
Line 5 B - D	Total	12	8	96.0			451
	window/door	2.5	7	17.5	2	35	
	typ int			78.5	5.3	416	
Line 5 A - B	Total	9.9	8	79.2	12.5	990	990
	typ ext						
Line 6	Total	22	8	176.0			1,248
	window/door	16	7	112.0	4	448	
	typ ext			64.0	12.5	800	
					Total 1st floor walls		**11,249**
Total Building Weight							**63,627**

at end unit add stucco to ext wall							
Line A 2-4	ext	24.2	20	484.0	8.8	4259	4,259
Line A 5 - 6	ext	19.2	20	384.0	8.8	3379	3,379
Roof		60	1	60.0	14	840	840
					Add @ end unit		8,478
Total Building Weight (end unit)							**72,105**

5.10 Description of RUAUMOKO Data Files

The three RUAMOKO data files corresponding to the poor-quality, typical-quality and superior-quality variants of the townhouse index building are included in the CD-ROM accompanying this report. These data files are self-contained and include, as ground motion input, one component of the acceleration time-history recorded at Canoga Park during the 1994 Northridge Earthquake. This ground motion is oriented along the short side of the building.

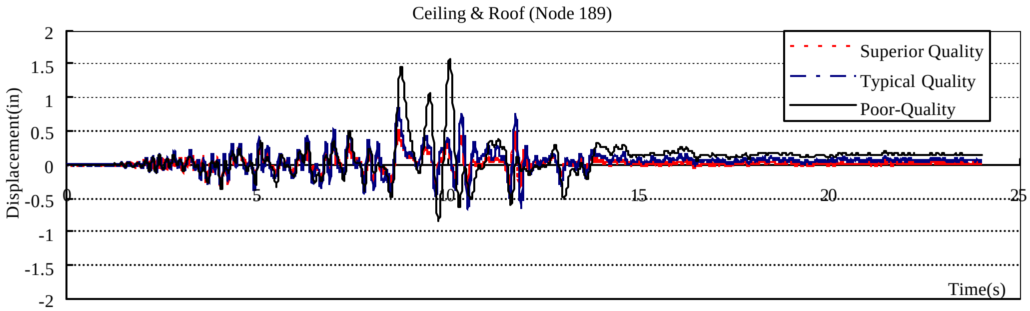
5.11 Analysis Examples

In this section, the three RUAUMOKO data files are used to evaluate the seismic response of the three variants of the townhouse index building when excited parallel to the short side of the building (y-axis direction) by the Canoga Park record of the 1994 Northridge Earthquake scale to a Peak Ground Acceleration (PGA) of 0.50 g.

Table 5.5 shows the fundamental frequency computed based on the initial stiffness for each of the construction variants of the townhouse index building. Figure 5.9 shows, for the three construction variants of the townhouse index building, the displacement time-histories in the y-direction at the second floor and roof level, respectively. Figure 5.10 presents, for the three construction variants, the hysteretic response of the OSB shear walls along wall line 6 of the townhouse index building. The graph on the left hand side represents the behavior of the first floor wall (element 403) and the graph on the right hand side represents the behavior of the second floor wall (element 486). The deformation is concentrated in the first floor shear walls in the all construction variants. For the poor-quality variant, the maximum displacement reaches a drift value of 1.6%. On the other hand, the drift of the superior quality variant remains around 0.5%.

Table 5.5 Fundamental Frequencies of Town-Index Building.

Construction Variant	Fundamental Frequency (Hz)	Mode of Vibration
Poor-Quality	5.68	Y-direction
Typical-Quality	6.22	Y-direction
Superior-Quality	6.68	Y-direction



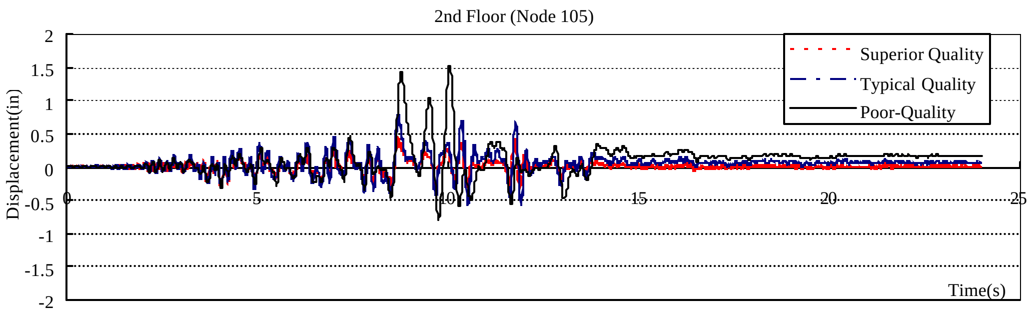


Figure 5.9 Displacement Time-Histories in the Y-Direction for Townhouse Index Building Under Canoga Park Record, PGA = 0.50 g.

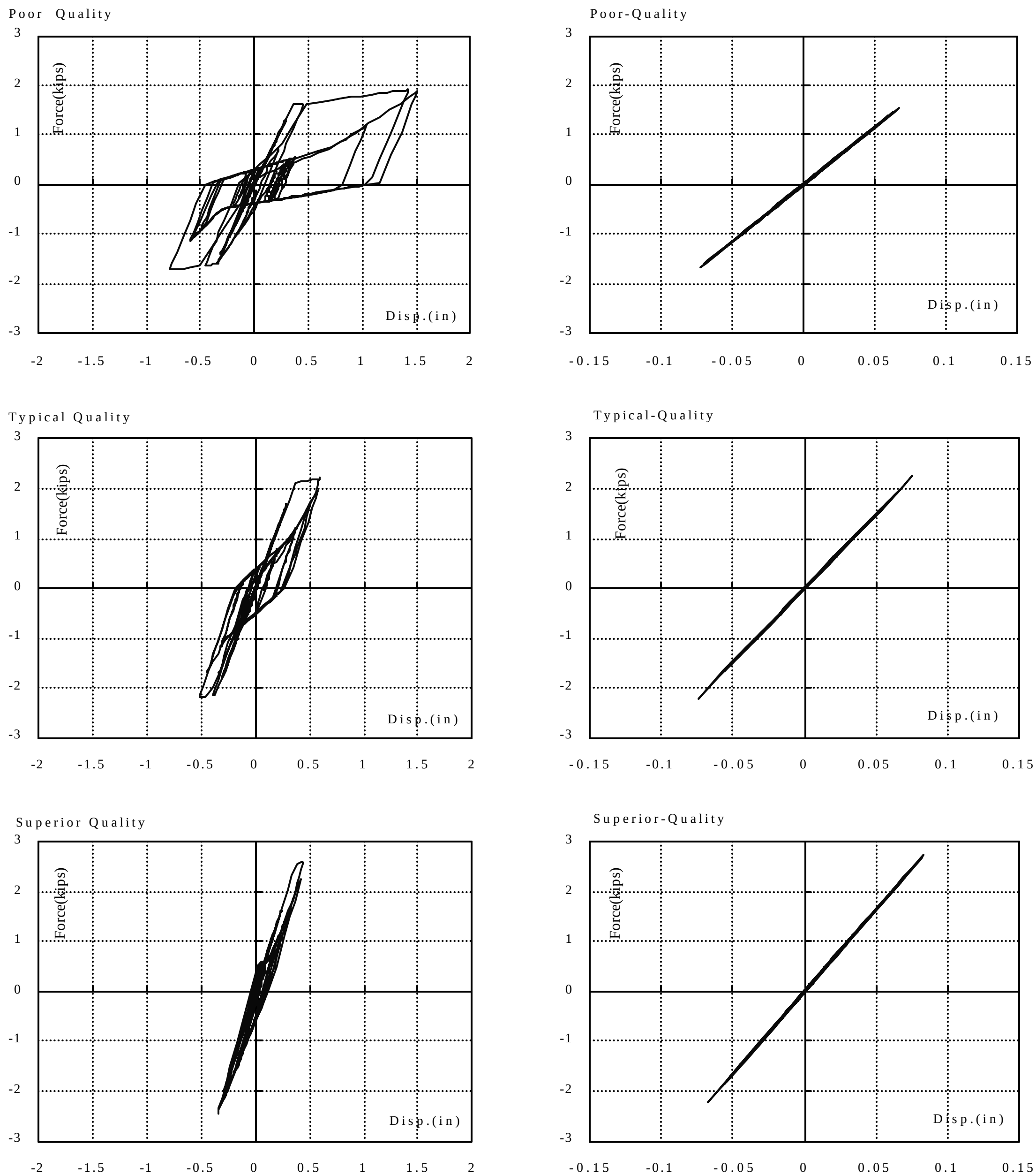
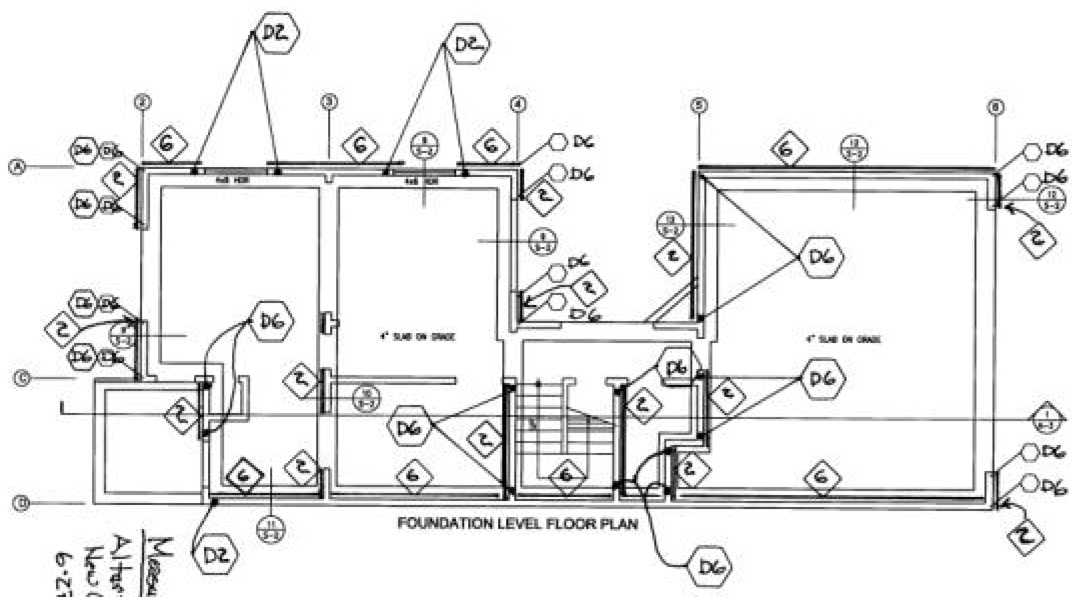


Figure 5.10 Hysteresis Loops of Walls Along Wall Line 6 (Element 403 on the left side and 486 on the right side) for Townhouse Index Building Under Canoga Park Record, PGA = 0.50g.

5.12 Retrofit of Townhouse Index Building: Retrofit Measure No. 5

The retrofit measure No. 5 represents an alternate new construction for the townhouse index building. It is geared towards greatly reducing the first floor drift in an attempt to reduce damage losses. New shear walls are added on the first floor and the nail schedules of the existing shear walls are modified, as shown in Figure 5.11 for a typical unit of the building. It is intended that the second floor of the building remain per the original design. Again, the CASHEW program computed the properties of these new and modified shear walls. The locations, orientation and numbering of the shear elements for the first floor interior and exterior vertical shear walls retrofitted according to measure No. 5 are given in Figure 5.12. The parameters of the Wayne Stewart hysteresis rule used to model these vertical shear walls are given in Table 5.6.



SHEAR WALL SCHEDULE								
SYMBOL	ALLOWABLE SHEAR (PLF)	SHEATHING MATERIAL	MIN STUD AT ADJOINING PANEL EDGES	FOUNDATION SILL	SHEATHING EDGE NAILS (1)	SHEATHING INTERMEDIATE NAILS (1)	ANCHOR BOLTS (2)	
6		15/32 STR I Rated Plwd Sheathing	2x	3x	8d@6"	8d@12"	~~1/2"@48"~~	5/8@ 48"
4								
3								
2			3x	3x	8d@2"		~~1/2"@16"~~	5/8@ 16"
S1								
S2								
G1								

1. Common nails, substitutions must be approved by Engineer.
2. Minimum two bolts per piece of sill.

Figure 5.11 Modifications to First Floor OSB Shear Walls for Retrofit Measure No.5 of Townhouse Index Building (Cobeen, 2001).

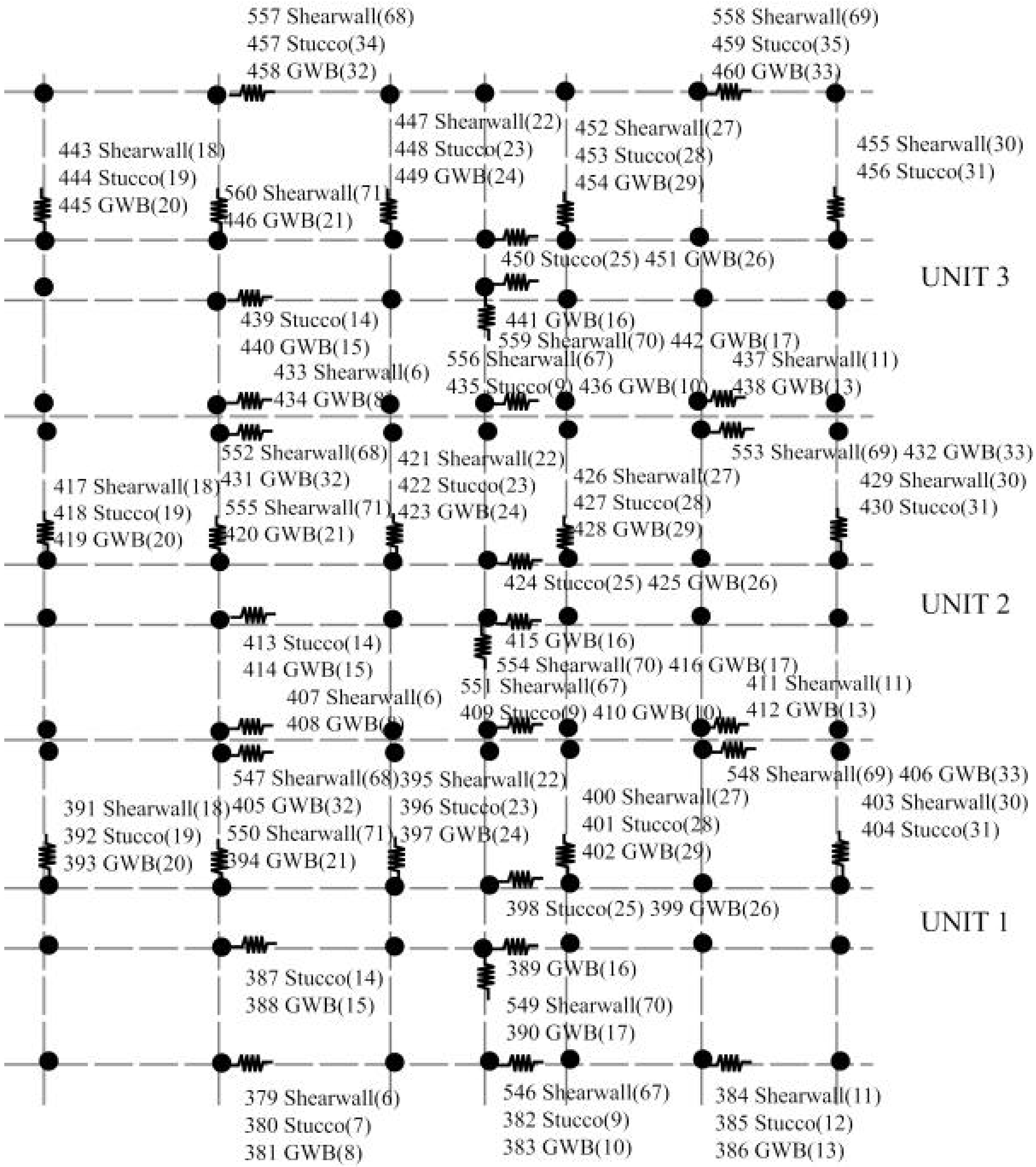


Figure 5.12 Locations, Orientation and Numbering of First Floor Shear Elements for Retrofit Measure No. 5 of the Townhouse Index Building.

Table 5.6 Parameters of Wayne Stewart Hysteresis Rule for Shear Walls of the Retrofit Measure No. 5 of the Townhouse Index Building.

Wall Type	Story	Wall Line	Shear Element No	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Shearwall OSB	1st	LineD 2-4	6	7.16	26.51	0.072	11.69	1.75	-0.066	1.16	0.75	1.09
		Line D 4-5	67	4.21	15.60	0.072	6.88	1.03	-0.066	1.16	0.75	1.09
		Line D 5-6	11	6.92	25.62	0.072	11.30	1.70	-0.066	1.16	0.75	1.09
		Line A 2-4	68	5.83	21.61	0.072	9.53	1.43	-0.066	1.16	0.75	1.09
		Line A5-6	69	6.92	25.62	0.072	11.30	1.70	-0.066	1.16	0.75	1.09
		Line2	18	16.70	42.77	0.083	26.99	3.91	-0.094	1.00	0.77	1.08
		Line3	71	4.79	12.26	0.083	7.74	1.12	-0.094	1.00	0.77	1.08
		Line4	22	12.70	32.53	0.083	20.53	2.97	-0.094	1.00	0.77	1.08
		Line45	70	8.53	21.83	0.083	13.78	2.00	-0.094	1.00	0.77	1.08
		Line5	27	29.23	74.85	0.083	47.23	6.84	-0.094	1.00	0.77	1.08
		Line6	30	5.22	13.37	0.083	8.44	1.23	-0.094	1.00	0.77	1.08

The retrofit measure No. 5 was applied to the typical-quality variant of the townhouse index building. As an example analysis, townhouse index building retrofitted with measure No. 5 was excited along its short side by the Canoga Park record of the 1994 Northridge Earthquake scale to a Peak Ground Acceleration (PGA) of 0.50 g. The data file for this case is included in the CD-ROM accompanying this report.

Table 5.7 compares the fundamental frequency computed based on the initial stiffness for the typical-quality construction variant townhouse index building retrofitted according to measure No. 5 with that of the original building. Figure 5.13 presents the resulting displacement time-histories at the first floor and roof level, while Figure 5.14 shows the hysteretic responses of the vertical walls along wall line 6. A comparison of these results with the results shown in Figures 5.9 and 5.10 indicates that the retrofit measure No. 5 is an effective method to reduce the seismic response of the townhouse index building.

Table 5.7 Fundamental Frequencies of the Townhouse Index Building Retrofitted with Measure No. 5.

Construction Variant	Fundamental Frequency (Hz)	Mode of Vibration
Retrofit Measure #5	7.07	Y-direction
Typical Quality	6.22	Y-direction

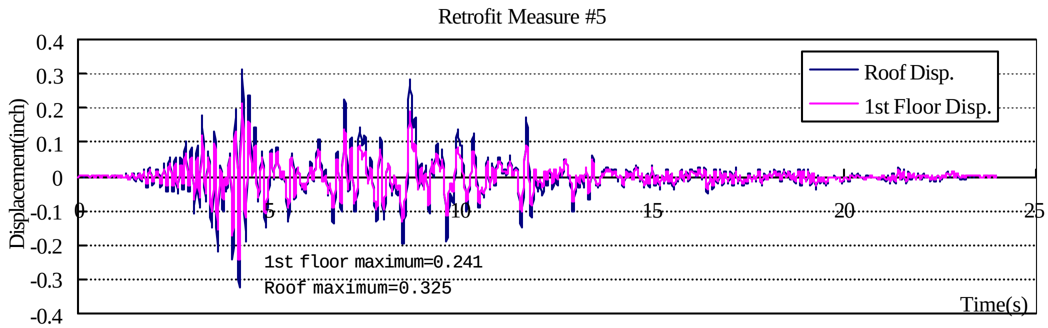


Figure 5.13 Displacement Time-Histories in the Y-Direction for Townhouse Index Building Retrofitted with Measure No. 5.

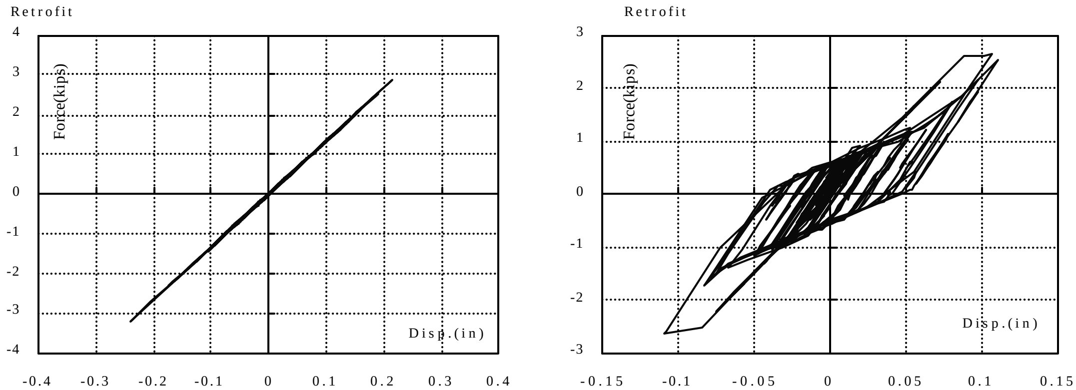


Figure 5.14 Hysteresis Loops of Shear Walls along Wall Line 6 (Element 403 on the left side and 486 on the right side) for Townhouse Index Building Retrofitted with Measure No. 5.

6. MODELING OF INDEX BUILDING 4: APARTMENT BUILDING

6.1 General Description

The fourth index building considered in this study represents a three-story multi-family apartment building with attached-car garages on the ground level. It is intended to have built in the 1960's and located in either Northern or Southern California. The design is based on partially engineered construction. In particular, the unit shears in plywood shear walls have been checked in accordance with the 1964 edition of the Uniform Building Code. To the extend possible, characteristic materials and fastening have been identified. Figure 6.1 shows plan views of the parking level of the building along with a typical floor plan. The architectural and structural drawings of the apartment building index building are included in Appendix D (Cobeen 2001).

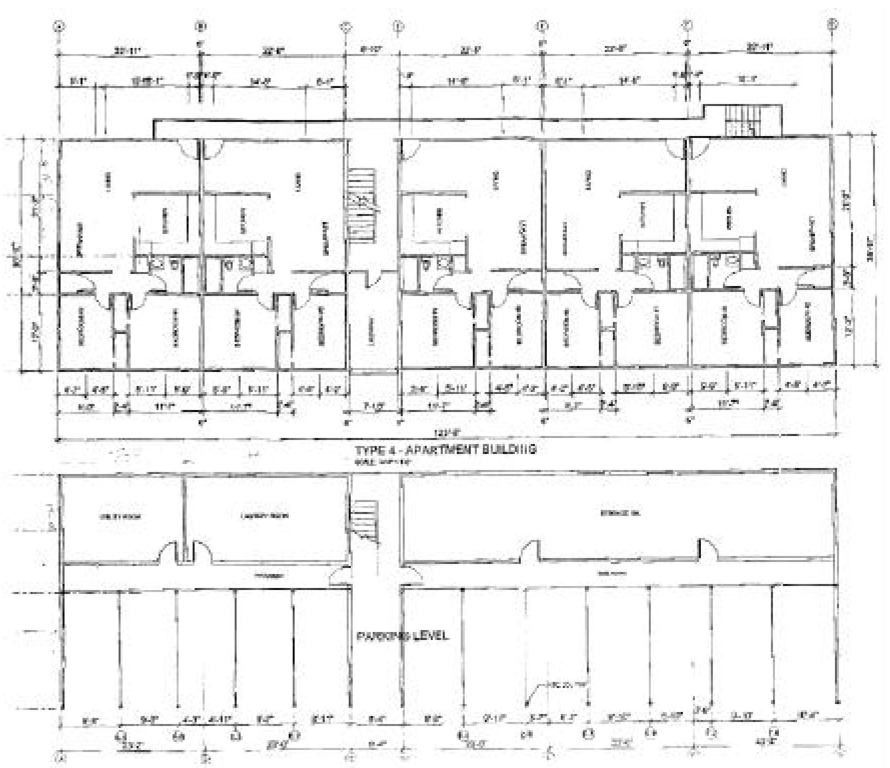


Figure 6.1 Plan Views of Apartment Index Building (Cobeen 2001).

The exterior walls of the apartment index building are sheathed with stucco (UBC Tables 25B and 25C) on the outside along with wood shear wall (3/8 in thick Struct I Plywood) and gypsum wallboard (1/2 in thick) on the inside. Furring nails (3/8-in head), spaced at 6 in on center along the vertical studs, are used to attach the wire mesh of the stucco finish to the wood framing.

Eight-penny common nails spaced at 6, 4 or 3 in along the edges and 12 in on the field are used to attach the OSB panels to the framing. All interior gypsum walls are sheathed on both sides. Drywall nails (1-5/8 in long) spaced at 7 in on center along the vertical studs (spaced at 16-in on center) are used to attach all gypsum walls to the framing. The gypsum walls are assumed to be positioned vertically.

The floor diaphragm of the apartment building is composed of 2x12 joists spaced at 16 in on center and 5/8" T&G PLWD or OSB sheathing. The apartment index building is supported on a slab on grade. The roof diaphragm is built with 2x6 joists spaced at 24 in on center and ½" PLWD. The ceiling is made of 5/8 in thick gypsum wallboard.

6.2 Description of Construction Variants

Three construction variants are defined for the apartment index building. The three variants represent a poor-quality, typical-quality, superior-quality construction, respectively. The characteristics of each construction variant are the same as that of the large-house index building (see section 4.3).

6.3 Modeling Assumptions

The main assumptions used to develop the numerical pancake model of each construction variant of the apartment index building are briefly discussed in this section.

Each stucco wall, plywood shear wall and gypsum wall is modeled by an in-plane shear element exhibiting the Wayne Stewart hysteresis rule, as described in Chapter 2. When exterior walls are sheathed with stucco and plywood on the outside and gypsum on the inside, three parallel shear elements are used to simulate the in-plane behavior of the three sheathing materials. Interior walls consist of gypsum wallboards and some plywood shear walls. For combined plywood and gypsum walls, two parallel shear elements are used as for the exterior walls. In case of the wall sheathed with gypsum on both sides, only one shear element per wall line is considered. The in-plane behavior of the floor diaphragm is modeled by linear-elastic quadrilateral finite elements.

6.4 Node Numbering

The pancake model of the apartment-building index building incorporates four layers of nodes. Figure 6.2 shows the location, orientation and number assigned to the nodes on each floor.

The first floor nodes are located below the walls on the first story of the building. The earthquake ground motion is applied simultaneously at these nodes. The second floor nodes are located at the level of the second floor diaphragm and are used to connect the shear elements representing the walls on the first floor from the first floor level to the second floor diaphragm and to connect also the shear elements representing the vertical walls between the second floor diaphragm and the third floor level. Similarly, third floor nodes are located at the level of the third floor. The ceiling nodes are used to connect the interior and exterior shear elements representing the vertical walls on the third floor.

6.5 Elements Description and Location

Sixty-nine shear elements are used to represent the walls on the first floor of the apartment index building. Figure 6.3 shows the location, orientation and number assigned to each of these shear elements in the RUAUMOKO data files. The numbers in the bracket following each element number represent the properties of the elements. All units are identical except for the exterior stucco. The location, orientation and number assigned to the shear elements used to represent the walls on the second floor and third floor of the apartment building index building are also illustrated in Figure 6.3.

The location, orientation and number assigned to linear-elastic quadrilateral finite elements used to represent the floors and ceiling diaphragms of the apartment building index building are illustrated in Figure 6.4.

The location, orientation and number assigned to linear-elastic frame elements used along the edges of the floors and ceiling diaphragms to connect the corners of the quadrilateral finite elements to vertical wall elements are illustrated in Figure 6.5.

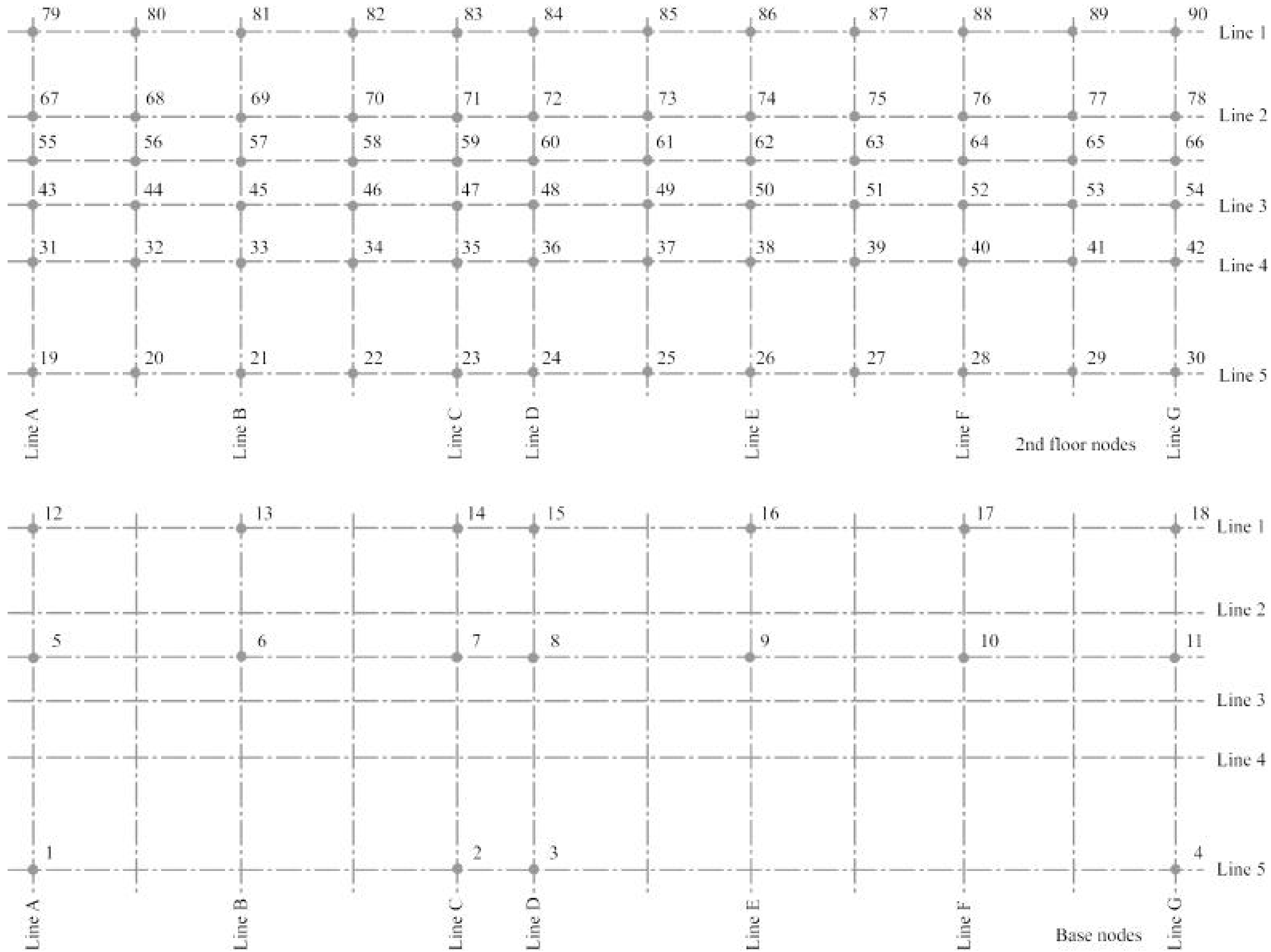


Figure 6.2 Node Numbering for Pancake Model of Apartment Index Building.

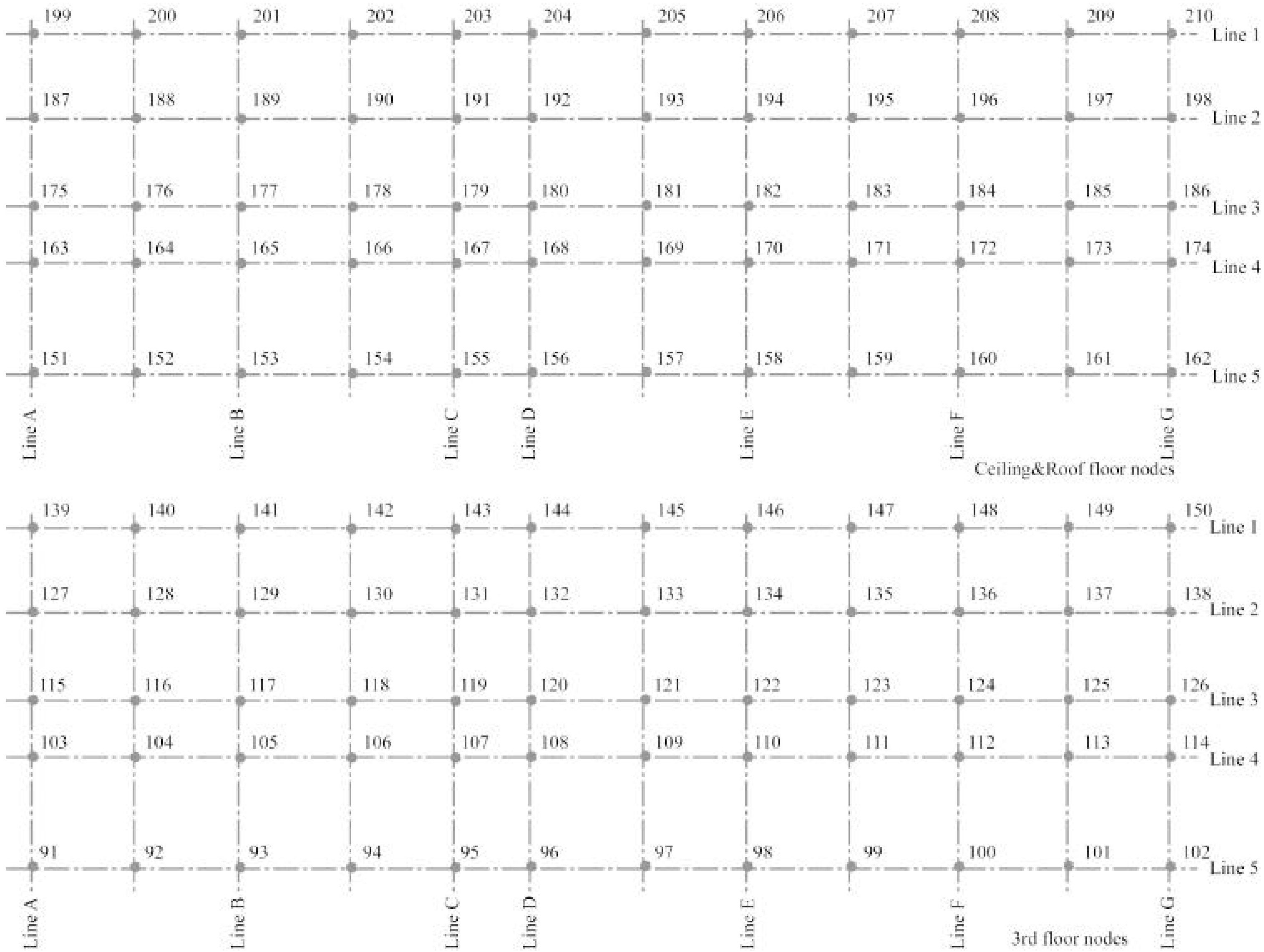


Figure 6.2 Node Numbering for Pancake Model of Apartment Index Building (continued).

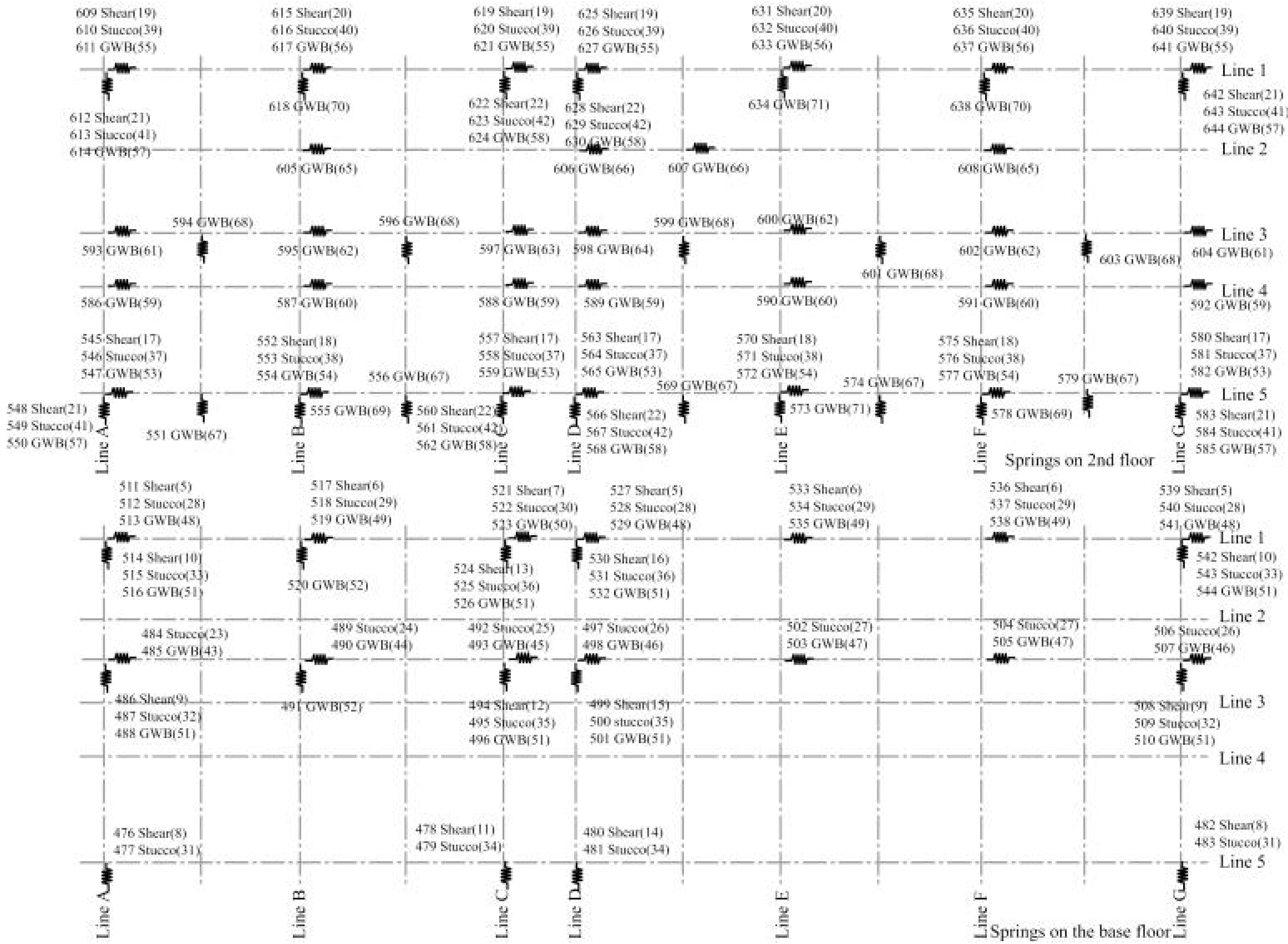


Figure 6.3 Locations, Orientation and Numbering of Shear Element for Interior and Exterior Vertical Walls on the First Floor and the Second Floor of the Apartment Index Building

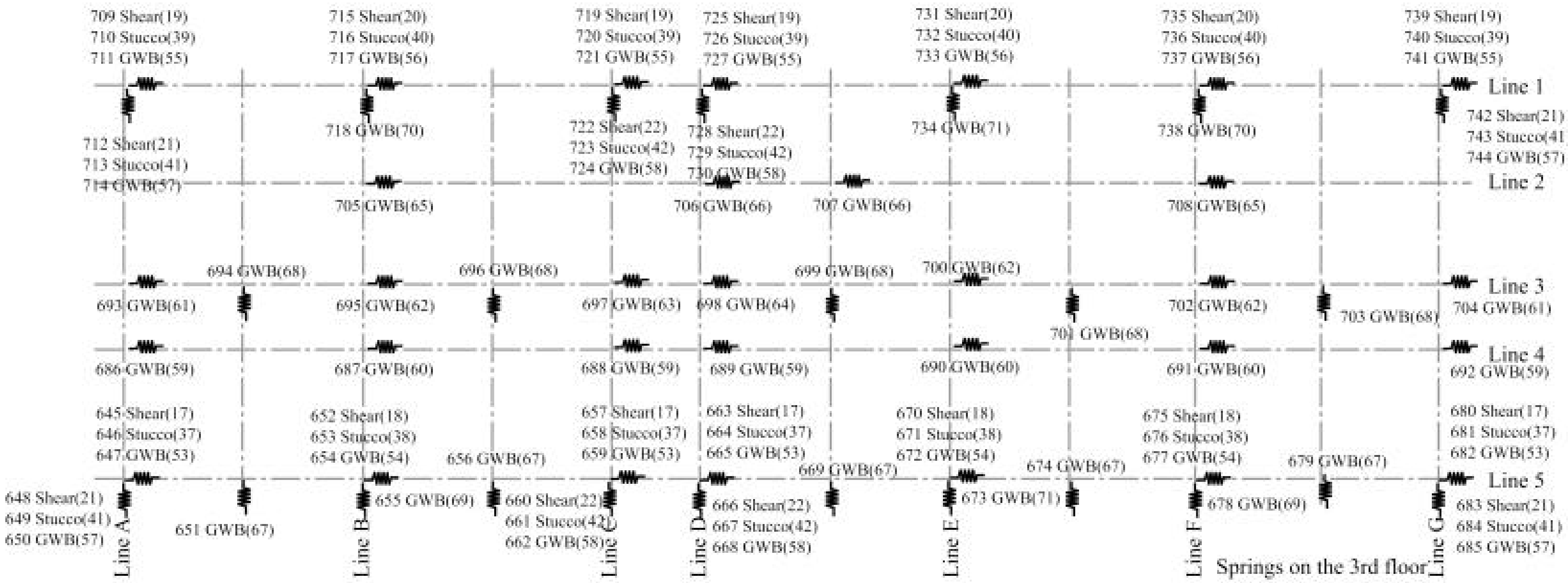


Figure 6.3 Locations, Orientation and Numbering of Shear Element for Interior and Exterior Vertical Walls on the Third Floor of the Apartment Index Building (continued).

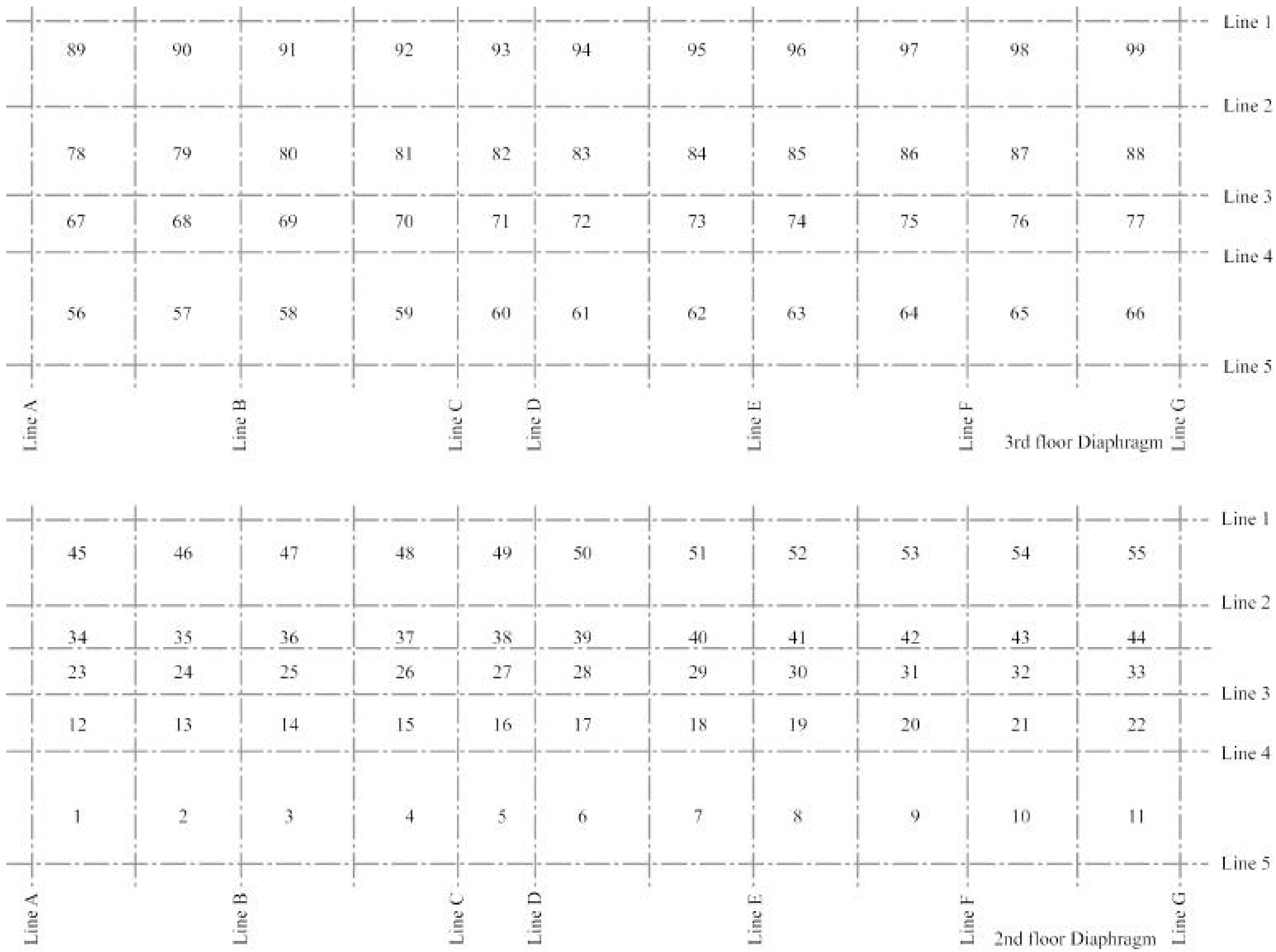


Figure 6.4 Locations, Orientation and Numbering of Quadrilateral Elements of the Apartment Index Building.

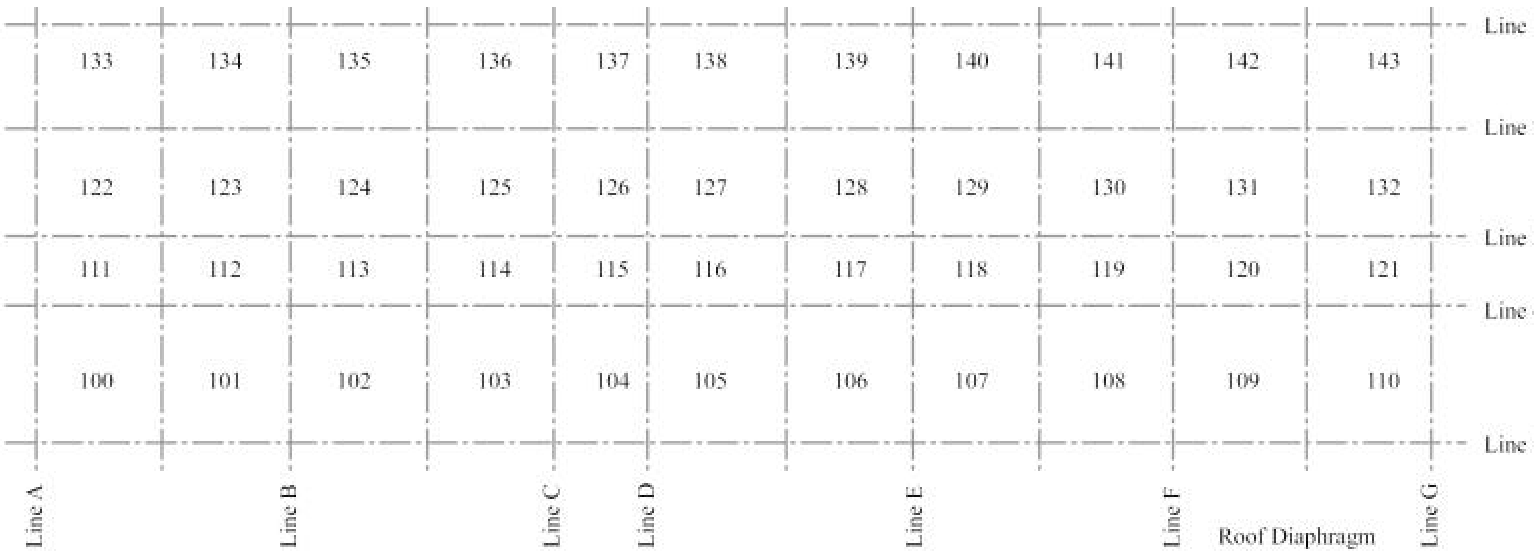


Figure 6.4 Locations, Orientation and Numbering of Quadrilateral Elements of the Apartment Index Building (continued).

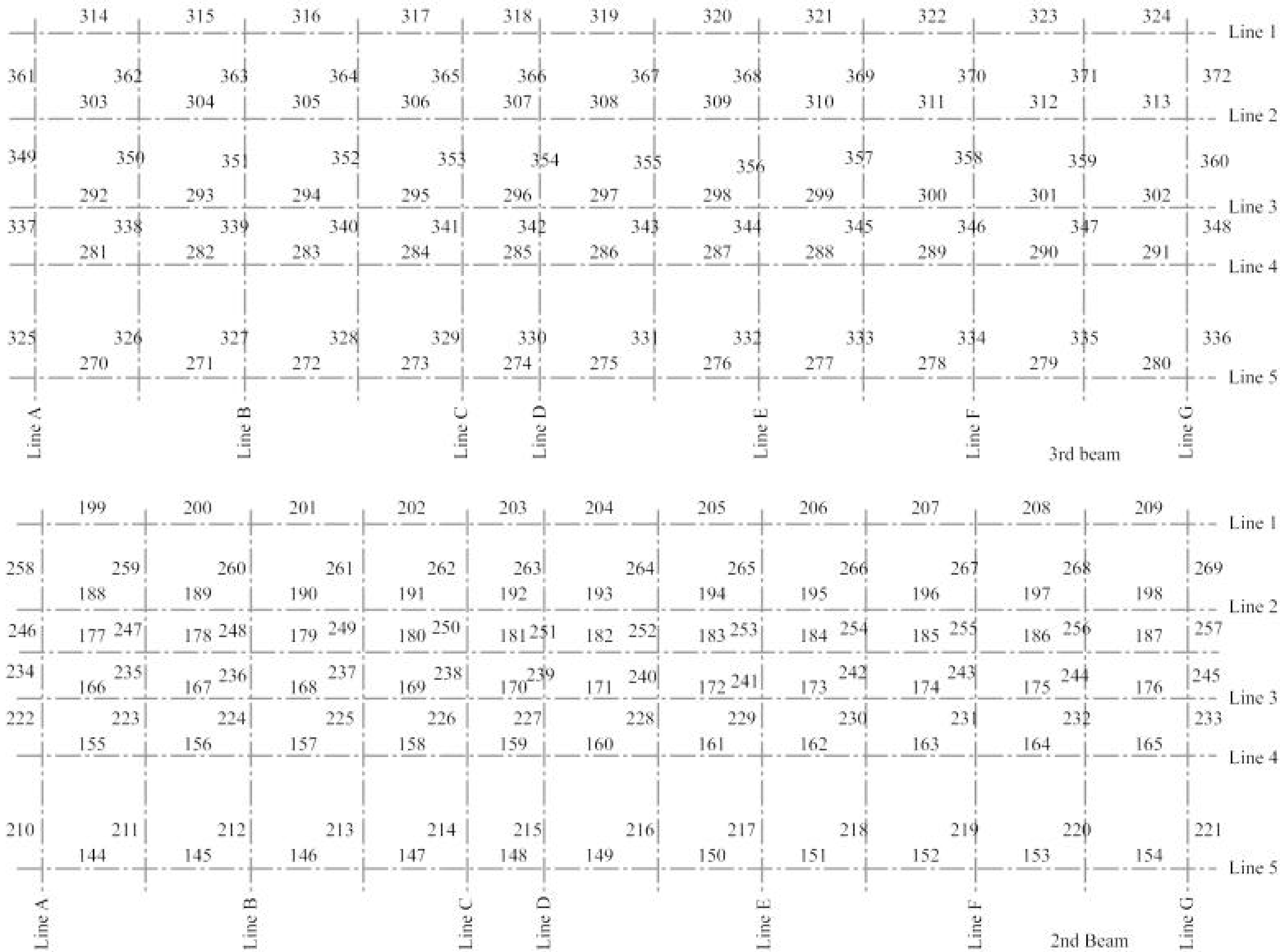


Figure 6.5 Locations, Orientation and Numbering of Frame Elements of the Apartment Index Building.

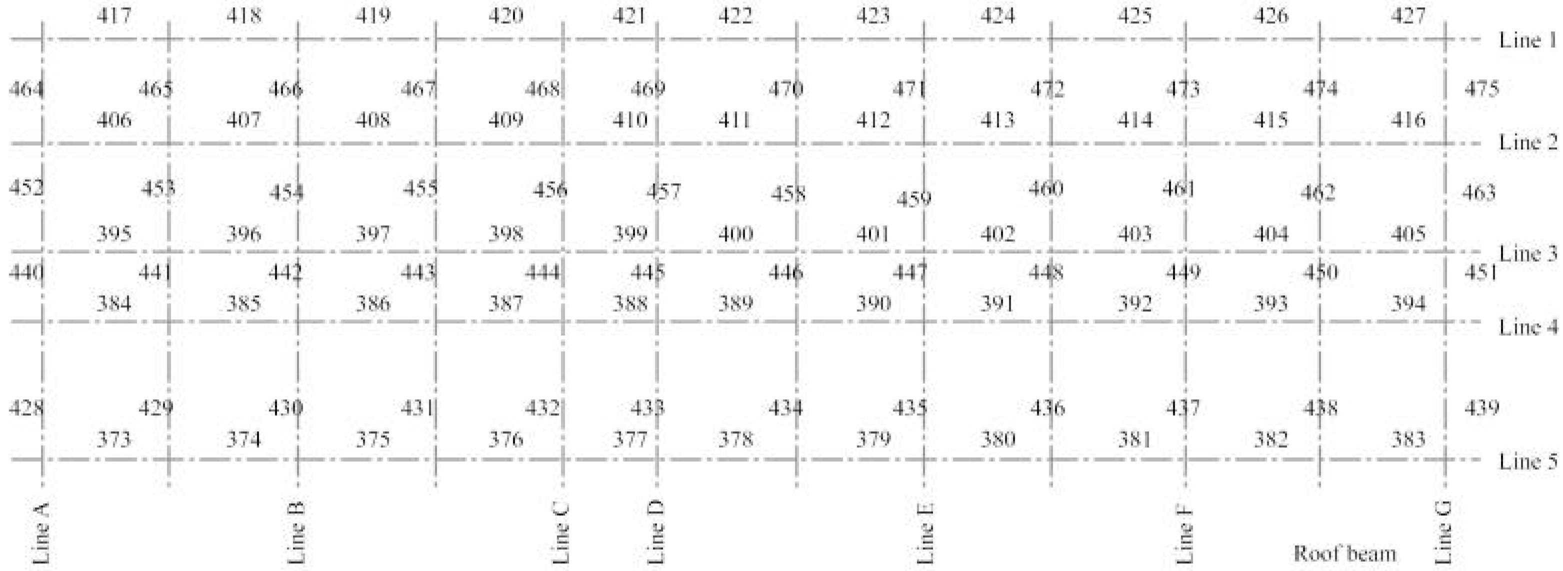


Figure 6.5 Locations, Orientation and Numbering of Frame Elements of the Apartment Index Building (continued).

6.6 Hysteretic Parameters for Vertical Wall Shear Elements

The in-plane cyclic responses of the vertical walls incorporated in the apartment building index building were modeled by shear elements exhibiting the same Wayne Stewart Hysteresis law as the other index buildings. The hysteretic parameters for each stucco and gypsum wall were estimated from available cyclic test data on wall assemblies. The hysteretic parameters for the plywood shearwalls were computed by the computer program CASHEW: Cyclic Analysis of wood SHEar Walls (Folz and Filiatrault, 2000) developed under Task 1.5.1 of the CUREE-Caltech Woodframe Project.

These tests of stucco and gypsum were conducted at the University of California at Irvine for the City of Los Angeles (COLA) Project (Pardoen, 2000). The parameter of the Wayne Stewart hysteresis of the stucco is the same as that of the small house-index building as shown in table 3.2. The parameters for 8-ft long segments of the gypsum walls are the same as that of the large-house index building. The properties of the plywood shear walls were predicted by the computer program CASHEW: Cyclic Analysis of wood SHEar Walls (Folz and Filiatrault, 2000). All hysteretic properties for the spring elements representing the plywood shear walls were obtained by adjusting the strength and stiffness values for the actual length of full wall piers in each wall line. The resulting properties are shown in Tables 6.1, 6.2, and 6.3 for the poor-quality, typical-quality and superior-quality variant, respectively.

Table 6.1 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Poor-Quality Variant of the Apartment Index Building.

Wall Type	Story	Location	Direction	Shear Element No.	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Shear wall PLWD	1st	A1,D1,G1	X	5	5.81	19.22	0.077	9.46	1.40	5.000	5.81	19.22	0.08
		B1,E1,F1	X	6	9.60	31.75	0.077	15.62	2.31	6.000	9.60	31.75	0.08
		C1	X	7	3.79	12.54	0.077	6.17	0.91	7.000	3.79	12.54	0.08
		A5,G5	Y	8	4.61	16.11	0.074	7.52	1.12	8.000	4.61	16.11	0.07
		A23,G23	Y	9	7.38	25.79	0.074	12.04	1.79	9.000	7.38	25.79	0.07
		A1,G1	Y	10	2.78	9.69	0.074	4.53	0.68	10.000	2.78	9.69	0.07
		C5	Y	11	4.01	14.01	0.074	6.54	0.97	11.000	4.01	14.01	0.07
		C23	Y	12	6.78	23.69	0.074	11.06	1.65	12.000	6.78	23.69	0.07
		C1	Y	13	2.78	9.69	0.074	4.53	0.68	13.000	2.78	9.69	0.07
		D5	Y	14	5.05	16.72	0.077	8.22	1.22	14.000	5.05	16.72	0.08
		D23	Y	15	8.55	28.27	0.077	13.91	2.05	15.000	8.55	28.27	0.08
		D1	Y	16	3.50	11.56	0.077	5.69	0.84	16.000	3.50	11.56	0.08
	2nd,3rd	A5,C5,D5,G5	X	17	1.96	6.77	0.073	3.19	0.48	17.000	1.96	6.77	0.07
		B5,E5,F5	X	18	3.92	13.54	0.073	6.38	0.95	18.000	3.92	13.54	0.07
		A1,C1,D1,G1	X	19	1.83	6.32	0.073	2.98	0.45	19.000	1.83	6.32	0.07
		B1,E1,F1	X	20	3.66	12.64	0.073	5.95	0.89	20.000	3.66	12.64	0.07
		A5,G5,A1,G1	Y	21	4.29	14.82	0.073	6.98	1.04	21.000	4.29	14.82	0.07
		C5,D5,C1,D1	Y	22	4.81	16.63	0.073	7.83	1.16	22.000	4.81	16.63	0.07
Vertical Exterior Stucco	1st	A23	X	23	1.45	16.59	0.082	2.17	0.37	23.000	1.45	16.59	0.08
		B23	X	24	3.40	39.02	0.082	5.09	0.85	24.000	3.40	39.02	0.08
		C23	X	25	1.96	22.44	0.082	2.93	0.49	25.000	1.96	22.44	0.08
		D23,G23	X	26	1.70	19.51	0.082	2.55	0.43	26.000	1.70	19.51	0.08
		E23,F23	X	27	3.65	41.95	0.082	5.48	0.92	27.000	3.65	41.95	0.08
		A1,D1,G1	X	28	1.96	22.44	0.082	2.93	0.49	28.000	1.96	22.44	0.08
		B1,E1,F1	X	29	3.23	37.07	0.082	4.84	0.81	29.000	3.23	37.07	0.08
		C1	X	30	1.28	14.64	0.082	1.91	0.32	30.000	1.28	14.64	0.08
		A5,G5	Y	31	1.96	22.44	0.082	2.93	0.49	31.000	1.96	22.44	0.08
		A23,G23	Y	32	3.13	35.94	0.082	4.69	0.79	32.000	3.13	35.94	0.08
		A1,G1	Y	33	1.18	13.50	0.082	1.76	0.30	33.000	1.18	13.50	0.08
		C5,D5	Y	34	1.70	19.51	0.082	2.55	0.43	34.000	1.70	19.51	0.08

		C23,D23	Y	35	2.87	33.01	0.082	4.31	0.72	35.000	2.87	33.01	0.08
		C1,D1	Y	36	1.18	13.50	0.082	1.76	0.30	36.000	1.18	13.50	0.08
	2nd,3rd	A5,C5,D5,G5	X	37	1.28	14.64	0.082	1.91	0.32	37.000	1.28	14.64	0.08
		B5,E5,F5	X	38	2.55	29.27	0.082	3.82	0.64	38.000	2.55	29.27	0.08
		A1,C1,D1,G1	X	39	1.19	13.66	0.082	1.79	0.30	39.000	1.19	13.66	0.08
		B1,E1,F1	X	40	2.38	27.32	0.082	3.57	0.60	40.000	2.38	27.32	0.08
		A5,G5,A1,G1	Y	41	2.79	32.03	0.082	4.18	0.70	41.000	2.79	32.03	0.08
		C5,D5,C1,D1	Y	42	3.13	35.94	0.082	4.69	0.79	42.000	3.13	35.94	0.08
Vertical Gypsum	1st	A23	X	43	1.01	15.15	0.064	1.37	0.26	43.000	1.01	15.15	0.06
		B23	X	44	2.37	35.63	0.064	3.22	0.60	44.000	2.37	35.63	0.06
		C23	X	45	1.37	20.49	0.064	1.85	0.35	45.000	1.37	20.49	0.06
		D23,G23	X	46	1.19	17.82	0.064	1.61	0.30	46.000	1.19	17.82	0.06
		E23,F23	X	47	2.55	38.30	0.064	3.46	0.64	47.000	2.55	38.30	0.06
		A1,D1,G1	X	48	1.37	20.49	0.064	1.85	0.35	48.000	1.37	20.49	0.06
		B1,E1,F1	X	49	2.25	33.85	0.064	3.06	0.57	49.000	2.25	33.85	0.06
		C1	X	50	0.89	13.36	0.064	1.21	0.23	50.000	0.89	13.36	0.06
		A23,C23,D23,G23,A1,C1,D1,G1	Y	51	0.82	12.33	0.064	1.12	0.21	51.000	0.82	12.33	0.06
		B23,B1	Y	52	1.64	24.65	0.064	2.23	0.41	52.000	1.64	24.65	0.06
	2nd,3rd	A5,C5,D5,G5	X	53	0.89	13.36	0.064	1.21	0.23	53.000	0.89	13.36	0.06
		B5,E5,F5	X	54	1.78	26.72	0.064	2.41	0.45	54.000	1.78	26.72	0.06
		A1,C1,D1,G1	X	55	0.83	12.47	0.064	1.13	0.21	55.000	0.83	12.47	0.06
		B1,E1,F1	X	56	1.66	24.94	0.064	2.25	0.42	56.000	1.66	24.94	0.06
		A5,G5,A1,G1	Y	57	1.95	29.25	0.064	2.64	0.49	57.000	1.95	29.25	0.06
		C5,D5,C1,D1	Y	58	8.72	131.22	0.064	11.83	2.18	58.000	8.72	131.22	0.06
		A4,C4,D4,G4	X	59	2.81	42.16	0.064	3.80	0.71	59.000	2.81	42.16	0.06
		B4,E4,F4	X	60	4.03	60.57	0.064	5.46	1.01	60.000	4.03	60.57	0.06
		A3,G3	X	61	2.02	30.29	0.064	2.73	0.51	61.000	2.02	30.29	0.06
		B3,E3,F3	X	62	4.97	74.82	0.064	6.75	1.25	62.000	4.97	74.82	0.06
		C3	X	63	3.16	47.50	0.064	4.29	0.79	63.000	3.16	47.50	0.06
		D3	X	64	2.49	37.41	0.064	3.38	0.63	64.000	2.49	37.41	0.06
		B2,F2	X	65	4.97	74.82	0.064	6.75	1.25	65.000	4.97	74.82	0.06
		D2,DE2	X	66	1.25	18.71	0.064	1.69	0.32	66.000	1.25	18.71	0.06
		AB5,BC5,DE5,EF5,FG5	Y	67	2.88	43.35	0.064	3.91	0.72	67.000	2.88	43.35	0.06
		AB3,BC3,DE3,EF3,FG3	Y	68	2.13	32.07	0.064	2.89	0.54	68.000	2.13	32.07	0.06

		B5,F5	Y	69	5.76	86.69	0.064	7.82	1.44	69.000	5.76	86.69	0.06
		B1,F1	Y	70	8.84	133.00	0.064	11.99	2.21	70.000	8.84	133.00	0.06
		E5,E1	Y	71	4.36	65.61	0.064	5.92	1.09	71.000	4.36	65.61	0.06

Table 6.2 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Typical-Quality Variant of the Apartment Index Building.

Wall Type	Story	Location	Direction	Shear Element No.	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Shear wall PLWD	1st	A1,D1,G1	X	5	7.58	21.02	0.079	12.32	1.83	-0.085	1.05	0.76	1.09
		B1,E1,F1	X	6	12.52	34.72	0.079	20.36	3.02	-0.085	1.05	0.76	1.09
		C1	X	7	4.94	13.71	0.079	8.04	1.20	-0.085	1.05	0.76	1.09
		A5,G5	Y	8	6.20	21.00	0.076	10.08	1.50	-0.074	1.10	0.76	1.09
		A23,G23	Y	9	9.92	33.63	0.076	16.14	2.40	-0.074	1.10	0.76	1.09
		A1,G1	Y	10	3.73	12.63	0.076	6.07	0.90	-0.074	1.10	0.76	1.09
		C5	Y	11	5.39	18.26	0.076	8.77	1.31	-0.074	1.10	0.76	1.09
		C23	Y	12	9.11	30.89	0.076	14.83	2.21	-0.074	1.10	0.76	1.09
		C1	Y	13	3.73	12.63	0.076	6.07	0.90	-0.074	1.10	0.76	1.09
		D5	Y	14	6.59	18.27	0.079	10.72	1.59	-0.085	1.05	0.76	1.09
		D23	Y	15	11.15	30.91	0.079	18.12	2.69	-0.085	1.05	0.76	1.09
		D1	Y	16	4.56	12.64	0.079	7.41	1.10	-0.085	1.05	0.76	1.09
	2nd,3rd	A5,C5,D5,G5	X	17	2.71	10.03	0.072	4.43	0.67	-0.066	1.16	0.75	1.09
		B5,E5,F5	X	18	5.41	20.05	0.072	8.85	1.33	-0.066	1.16	0.75	1.09
		A1,C1,D1,G1	X	19	2.53	9.36	0.072	4.13	0.62	-0.066	1.16	0.75	1.09
		B1,E1,F1	X	20	5.05	18.72	0.072	8.26	1.24	-0.066	1.16	0.75	1.09
		A5,G5,A1,G1	Y	21	5.92	21.95	0.072	9.68	1.45	-0.066	1.16	0.75	1.09
		C5,D5,C1,D1	Y	22	6.65	24.62	0.072	10.86	1.63	-0.066	1.16	0.75	1.09
Vertical Exterior Stucco	1st	A23	X	23	1.86	21.33	0.082	2.79	0.47	-0.064	1.45	0.38	1.09
		B23	X	24	4.37	50.17	0.082	6.55	1.10	-1.064	1.45	0.38	1.09
		C23	X	25	2.51	28.85	0.082	3.77	0.63	-2.064	1.45	0.38	1.09
		D23,G23	X	26	2.19	25.09	0.082	3.28	0.55	-3.064	1.45	0.38	1.09
		E23,F23	X	27	4.69	53.94	0.082	7.04	1.18	-4.064	1.45	0.38	1.09
		A1,D1,G1	X	28	2.51	28.85	0.082	3.77	0.63	-5.064	1.45	0.38	1.09
		B1,E1,F1	X	29	4.15	47.66	0.082	6.22	1.04	-6.064	1.45	0.38	1.09

		C1	X	30	1.64	18.82	0.082	2.46	0.41	-7.064	1.45	0.38	1.09
		A5,G5	Y	31	2.51	28.85	0.082	3.77	0.63	-0.064	1.45	0.38	1.09
		A23,G23	Y	32	4.02	46.20	0.082	6.03	1.01	-0.064	1.45	0.38	1.09
		A1,G1	Y	33	1.51	17.35	0.082	2.27	0.38	-0.064	1.45	0.38	1.09
		C5,D5	Y	34	2.19	25.09	0.082	3.28	0.55	-0.064	1.45	0.38	1.09
		C23,D23	Y	35	3.69	42.44	0.082	5.54	0.93	-0.064	1.45	0.38	1.09
		C1,D1	Y	36	1.51	17.35	0.082	2.27	0.38	-0.064	1.45	0.38	1.09
	2nd,3rd	A5,C5,D5,G5	X	37	1.64	18.82	0.082	2.46	0.41	-0.064	1.45	0.38	1.09
		B5,E5,F5	X	38	3.28	37.63	0.082	4.91	0.82	-0.064	1.45	0.38	1.09
		A1,C1,D1,G1	X	39	1.53	17.56	0.082	2.30	0.39	-1.064	1.45	0.38	1.09
		B1,E1,F1	X	40	3.06	35.12	0.082	4.59	0.77	-2.064	1.45	0.38	1.09
		A5,G5,A1,G1	Y	41	3.59	41.18	0.082	5.38	0.90	-0.064	1.45	0.38	1.09
		C5,D5,C1,D1	Y	42	4.02	46.20	0.082	6.03	1.01	-0.064	1.45	0.38	1.09
Vertical Gypsum	1st	A23	X	43	1.14	17.16	0.064	1.55	0.29	-0.042	1.45	0.38	1.09
		B23	X	44	2.69	40.38	0.064	3.64	0.68	-0.042	1.45	0.38	1.09
		C23	X	45	1.55	23.22	0.064	2.10	0.39	-0.042	1.45	0.38	1.09
		D23,G23	X	46	1.35	20.19	0.064	1.82	0.34	-0.042	1.45	0.38	1.09
		E23,F23	X	47	2.89	43.41	0.064	3.92	0.73	-0.042	1.45	0.38	1.09
		A1,D1,G1	X	48	1.55	23.22	0.064	2.10	0.39	-0.042	1.45	0.38	1.09
		B1,E1,F1	X	49	2.55	38.36	0.064	3.46	0.64	-0.042	1.45	0.38	1.09
		C1	X	50	1.01	15.15	0.064	1.37	0.26	-0.042	1.45	0.38	1.09
		A23,C23,D23,G23,A1,C1,D1,G1	Y	51	0.93	13.97	0.064	1.26	0.24	-0.042	1.45	0.38	1.09
		B23,B1	Y	52	1.86	27.93	0.064	2.52	0.47	-0.042	1.45	0.38	1.09
	2nd,3rd	A5,C5,D5,G5	X	53	1.01	15.15	0.064	1.37	0.26	-0.042	1.45	0.38	1.09
		B5,E5,F5	X	54	2.02	30.29	0.064	2.73	0.51	-0.042	1.45	0.38	1.09
		A1,C1,D1,G1	X	55	0.94	14.14	0.064	1.28	0.24	-0.042	1.45	0.38	1.09
		B1,E1,F1	X	56	1.88	28.27	0.064	2.55	0.47	-0.042	1.45	0.38	1.09
		A5,G5,A1,G1	Y	57	2.21	33.15	0.064	2.99	0.56	-0.042	1.45	0.38	1.09
		C5,D5,C1,D1	Y	58	9.88	148.72	0.064	13.41	2.47	-0.042	1.45	0.38	1.09
		A4,C4,D4,G4	X	59	3.18	47.78	0.064	4.31	0.80	-0.042	1.45	0.38	1.09
		B4,E4,F4	X	60	4.56	68.64	0.064	6.19	1.14	-0.042	1.45	0.38	1.09
		A3,G3	X	61	2.28	34.32	0.064	3.10	0.57	-0.042	1.45	0.38	1.09
		B3,E3,F3	X	62	5.64	84.79	0.064	7.65	1.41	-0.042	1.45	0.38	1.09
		C3	X	63	3.58	53.84	0.064	4.86	0.90	-0.042	1.45	0.38	1.09

		D3	X	64	2.82	42.40	0.064	3.83	0.71	-0.042	1.45	0.38	1.09
		B2,F2	X	65	5.64	84.79	0.064	7.65	1.41	-0.042	1.45	0.38	1.09
		D2,DE2	X	66	1.41	21.20	0.064	1.92	0.36	-0.042	1.45	0.38	1.09
		AB5,BC5,DE5,EF5,FG5	Y	67	3.27	49.13	0.064	4.43	0.82	-0.042	1.45	0.38	1.09
		AB3,BC3,DE3,EF3,FG3	Y	68	2.42	36.34	0.064	3.28	0.61	-0.042	1.45	0.38	1.09
		B5,F5	Y	69	6.53	98.25	0.064	8.86	1.64	-0.042	1.45	0.38	1.09
		B1,F1	Y	70	10.02	150.74	0.064	13.59	2.51	-0.042	1.45	0.38	1.09
		E5,E1	Y	71	4.94	74.36	0.064	6.71	1.24	-0.042	1.45	0.38	1.09

Table 6.3 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of the Superior-Quality Variant of the Apartment Index Building.

Wall Type	Story	Location	Direction	Shear Element No.	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Shear wall PLWD	1st	A1,D1,G1	X	5	9.15	28.26	0.079	14.83	2.19	-0.085	1.05	0.76	1.09
		B1,E1,F1	X	6	15.11	46.69	0.079	24.49	3.61	-0.085	1.05	0.76	1.09
		C1	X	7	5.97	18.43	0.079	9.67	1.43	-0.085	1.05	0.76	1.09
		A5,G5	Y	8	7.03	23.52	0.076	11.44	1.70	-0.078	1.10	0.76	1.09
		A23,G23	Y	9	11.25	37.67	0.076	18.32	2.73	-0.078	1.10	0.76	1.09
		A1,G1	Y	10	4.23	14.15	0.076	6.89	1.03	-0.078	1.10	0.76	1.09
		C5	Y	11	6.11	20.45	0.076	9.95	1.48	-0.078	1.10	0.76	1.09
		C23	Y	12	10.33	34.60	0.076	16.83	2.50	-0.078	1.10	0.76	1.09
		C1	Y	13	4.23	14.15	0.076	6.89	1.03	-0.078	1.10	0.76	1.09
		D5	Y	14	7.95	24.58	0.079	12.89	1.90	-0.085	1.05	0.76	1.09
		D23	Y	15	13.45	41.57	0.079	21.81	3.21	-0.085	1.05	0.76	1.09
		D1	Y	16	5.50	17.00	0.079	8.92	1.32	-0.085	1.05	0.76	1.09
	2nd,3rd	A5,C5,D5,G5	X	17	3.22	11.65	0.073	5.26	0.79	-0.068	1.16	0.75	1.09
		B5,E5,F5	X	18	6.44	23.29	0.073	10.51	1.58	-0.068	1.16	0.75	1.09
		A1,C1,D1,G1	X	19	3.01	10.87	0.073	4.91	0.74	-0.068	1.16	0.75	1.09
		B1,E1,F1	X	20	6.01	21.74	0.073	9.81	1.48	-0.068	1.16	0.75	1.09
		A5,G5,A1,G1	Y	21	7.05	25.49	0.073	11.50	1.73	-0.068	1.16	0.75	1.09
		C5,D5,C1,D1	Y	22	7.91	28.60	0.073	12.91	1.94	-0.068	1.16	0.75	1.09
Vertical Exterior	1st	A23	X	23	2.06	23.70	0.082	3.09	0.52	-0.064	1.45	0.38	1.09
		B23	X	24	4.85	55.75	0.082	7.27	1.22	-1.064	1.45	0.38	1.09

Stucco		C23	X	25	2.79	32.06	0.082	4.19	0.70	-2.064	1.45	0.38	1.09
		D23,G23	X	26	2.43	27.88	0.082	3.64	0.61	-3.064	1.45	0.38	1.09
		E23,F23	X	27	5.22	59.93	0.082	7.82	1.31	-4.064	1.45	0.38	1.09
		A1,D1,G1	X	28	2.79	32.06	0.082	4.19	0.70	-5.064	1.45	0.38	1.09
		B1,E1,F1	X	29	4.61	52.96	0.082	6.91	1.16	-6.064	1.45	0.38	1.09
		C1	X	30	1.82	20.91	0.082	2.73	0.46	-7.064	1.45	0.38	1.09
		A5,G5	Y	31	2.79	32.06	0.082	4.19	0.70	-0.064	1.45	0.38	1.09
		A23,G23	Y	32	4.47	51.33	0.082	6.70	1.12	-0.064	1.45	0.38	1.09
		A1,G1	Y	33	1.68	19.28	0.082	2.52	0.42	-0.064	1.45	0.38	1.09
		C5,D5	Y	34	2.43	27.88	0.082	3.64	0.61	-0.064	1.45	0.38	1.09
		C23,D23	Y	35	4.10	47.15	0.082	6.15	1.03	-0.064	1.45	0.38	1.09
		C1,D1	Y	36	1.68	19.28	0.082	2.52	0.42	-0.064	1.45	0.38	1.09
	2nd,3rd	A5,C5,D5,G5	X	37	1.82	20.91	0.082	2.73	0.46	-0.064	1.45	0.38	1.09
		B5,E5,F5	X	38	3.64	41.81	0.082	5.46	0.91	-0.064	1.45	0.38	1.09
		A1,C1,D1,G1	X	39	1.70	19.51	0.082	2.55	0.43	-1.064	1.45	0.38	1.09
		B1,E1,F1	X	40	3.40	39.02	0.082	5.09	0.85	-2.064	1.45	0.38	1.09
		A5,G5,A1,G1	Y	41	3.98	45.76	0.082	5.97	1.00	-0.064	1.45	0.38	1.09
		C5,D5,C1,D1	Y	42	4.47	51.33	0.082	6.70	1.12	-0.064	1.45	0.38	1.09
Vertical Gypsum	1st	A23	X	43	1.35	20.19	0.064	1.82	0.34	-0.042	1.45	0.38	1.09
		B23	X	44	3.16	47.50	0.064	4.29	0.79	-0.042	1.45	0.38	1.09
		C23	X	45	1.82	27.32	0.064	2.47	0.46	-0.042	1.45	0.38	1.09
		D23,G23	X	46	1.58	23.75	0.064	2.15	0.40	-0.042	1.45	0.38	1.09
		E23,F23	X	47	3.40	51.07	0.064	4.61	0.85	-0.042	1.45	0.38	1.09
		A1,D1,G1	X	48	1.82	27.32	0.064	2.47	0.46	-0.042	1.45	0.38	1.09
		B1,E1,F1	X	49	3.00	45.13	0.064	4.07	0.75	-0.042	1.45	0.38	1.09
		C1	X	50	1.19	17.82	0.064	1.61	0.30	-0.042	1.45	0.38	1.09
		A23,C23,D23,G23,A1,C1,D1,G1	Y	51	1.10	16.43	0.064	1.49	0.28	-0.042	1.45	0.38	1.09
		B23,B1	Y	52	2.19	32.86	0.064	2.97	0.55	-0.042	1.45	0.38	1.09
	2nd,3rd	A5,C5,D5,G5	X	53	1.19	17.82	0.064	1.61	0.30	-0.042	1.45	0.38	1.09
		B5,E5,F5	X	54	2.37	35.63	0.064	3.22	0.60	-0.042	1.45	0.38	1.09
		A1,C1,D1,G1	X	55	1.11	16.63	0.064	1.50	0.28	-0.042	1.45	0.38	1.09
		B1,E1,F1	X	56	2.21	33.25	0.064	3.00	0.56	-0.042	1.45	0.38	1.09
		A5,G5,A1,G1	Y	57	2.59	38.99	0.064	3.52	0.65	-0.042	1.45	0.38	1.09
		C5,D5,C1,D1	Y	58	11.63	174.96	0.064	15.77	2.91	-0.042	1.45	0.38	1.09

		A4,C4,D4,G4	X	59	3.74	56.21	0.064	5.07	0.94	-0.042	1.45	0.38	1.09
		B4,E4,F4	X	60	5.37	80.75	0.064	7.28	1.35	-0.042	1.45	0.38	1.09
		A3,G3	X	61	2.69	40.38	0.064	3.64	0.68	-0.042	1.45	0.38	1.09
		B3,E3,F3	X	62	6.63	99.75	0.064	9.00	1.66	-0.042	1.45	0.38	1.09
		C3	X	63	4.21	63.34	0.064	5.71	1.06	-0.042	1.45	0.38	1.09
		D3	X	64	3.32	49.88	0.064	4.50	0.83	-0.042	1.45	0.38	1.09
		B2,F2	X	65	6.63	99.75	0.064	9.00	1.66	-0.042	1.45	0.38	1.09
		D2,DE2	X	66	1.66	24.94	0.064	2.25	0.42	-0.042	1.45	0.38	1.09
		AB5,BC5,DE5,EF5,FG5	Y	67	3.84	57.80	0.064	5.21	0.96	-0.042	1.45	0.38	1.09
		AB3,BC3,DE3,EF3,FG3	Y	68	2.84	42.75	0.064	3.86	0.71	-0.042	1.45	0.38	1.09
		B5,F5	Y	69	7.68	115.59	0.064	10.42	1.92	-0.042	1.45	0.38	1.09
		B1,F1	Y	70	11.78	177.34	0.064	15.99	2.95	-0.042	1.45	0.38	1.09
		E5,E1	Y	71	5.82	87.48	0.064	7.89	1.46	-0.042	1.45	0.38	1.09

6.7 Properties of Horizontal Floor and Roof Diaphragms

The in-plane stiffness of the floor diaphragm, G_d, is taken to be 800 kips/in, while the corresponding value for the ceiling diaphragm is take to be 400 kips/in. These values are prescribed by the NEHRP Guidelines for Seismic Rehabilitation of Buildings (FEMA, 1997). Each linear-elastic diaphragm finite element is assigned elastic properties such that:

$$Gt = \frac{Et}{2(1+\boldsymbol{n})} = G_d$$

where G is the equivalent shear modulus, E is the equivalent elastic modulus, $\boldsymbol{n}$ is the equivalent Poisson's ratio and t is the thickness of the finite element.

6.8 Properties of frame elements

The frame elements to connect the shearwalls to the floor and roof diaphragms are assumed to be truss elements with high axial stiffness and negligible flexural stiffness.

6.9 Weight Distribution

Table 6.4 list the seismic weights considered for the apartment index building. These weights were distributed as nodal lumped seismic weights according to the tributary areas of the nodes.

Table 6.4 Weights for Apartment Index Building.

Item	Location		Length (ft)	Width or Height (ft)	Area (sq ft)	Unit Weight (psf)	Weight #	Total Weight #	No.	Weight x No.
Roof			124	37	4588	15	68,820	68,820	1	
Walls 3-R	Line 1-Ext	Total	116	8	928			11,085	1	11,085
Walls 2-3		window/door			160	4	640			
		typ exterior			768	13.6	10,445			
	Line 2-Int	Total	55	8	440			2,640	1	2,640
		open & door			0	2	0			
		typ interior			440	6	2,640			
	Line 3/3.5-In	Total	115	8	920			5,120	1	5,120
		open & door			100	2	200			
		typ interior			820	6	4,920			
	Line 4-Int	Total	115	8	920			4,800	1	4,800
		open & door			180	2	360			
		typ interior			740	6	4,440			
	Line 5-Ext	Total	116	8	928			11,469	1	11,469
		window/door			120	4	480			
		typ exterior			808	13.6	10,989			
	Line A-Ext	Total	36.8	8	294.4			3,889	4	15,555
	Also C,D,G	window/door			12	4	48			
		typ ext			282.4	13.6	3,841			
	Line -Int A.2+	Total	40	8	320			1,512	5	7,560
	A.4+A.6+A.8	open & door			102	2	204			
		typ interior			218	6	1,308			
	Line B-Int	Total	37	8	296			1,480	3	4,440
	Also E, F	open & door			0	6	0			
		typ interior			296	5	1,480			
Main Floor			37	116	4292	8.2	35194	35194		
Exterior Balcony			4	163	652	40.7	26536	26536		
Stairs 3rd			4	20	80	40	3200	3200		
Stairs 2nd			4	40	160	40	6400	6400		
Walls 1st-2nd	Line 1-Ext	Total	116	8	928			11,930	1	11,930
		open & door			72	4	288			
		typ exterior			856	13.6	11,642			
	Line 2.5-Int	Total	116	8	928			7,652	1	7,652
		open & door			100	2	200			
		typ interior			828	9	7,452			
	Line 5-Soffit	Total	124	2	248			3,373	1	3,373
		open & door			0	2	0			
		typ interior			248	13.6	3,373			

6.10 Description of RUAUMOKO Data Files

The three RUAMOKO data files corresponding to the poor-quality, typical-quality and superior-quality variants are included in the CD-ROM accompanying this report. These data files are self-contained and include, as ground motion input, one component of the acceleration time-history recorded at Canoga Park during the 1994 Northridge Earthquake. This ground motion is oriented along the short side of the building.

6.11 Analysis Examples

In this section, the three RUAUMOKO data files are used to evaluate the seismic response of the three variants of the apartment index building when excited parallel to the short side of the building (y-axis direction) by the Canoga Park record of the 1994 Northridge Earthquake scale to a Peak Ground Acceleration (PGA) of 0.50 g.

Table 6.5 shows the fundamental frequency computed based on the initial stiffness and maximum responses at the second floor and roof levels obtained for each of the construction variants. Figure 6.6 shows for the three variants the displacement time-histories in the y-direction at the second floor and ceiling level, respectively. Figure 6.7 shows the comparison of the displacement time histories along line A and G. Figure 6.8 presents for the three construction variants the hysteresis loops of the plywood shear walls along line G of the apartment index building. The graphs on the left hand side represent the behavior on the first floor (element 542) and the graphs on the right hand side represent the behavior on the second floor (element 642). The deformation is concentrated in the first floor walls in all of the construction variants. This indicates that the parking garages introduced a soft story. The response of line A is almost half of that of line G. For the poor-quality variant, the maximum displacement reaches a story drift value of 6% on the first story, and that of the superior quality variant remains around 4%.

Table 6.5 Fundamental Frequencies of Apartment Index Building.

Construction Variant	Fundamental Frequency (Hz)	Maximum Response – Node 210 (in)	Maximum Response - Node 199 (in)	Mode of Vibration
Poor-Quality	3.48	6.11	3.44	Y-direction
Typical-Quality	3.96	5.03	2.11	Y-direction
Superior-Quality	4.30	4.21	1.30	Y-direction

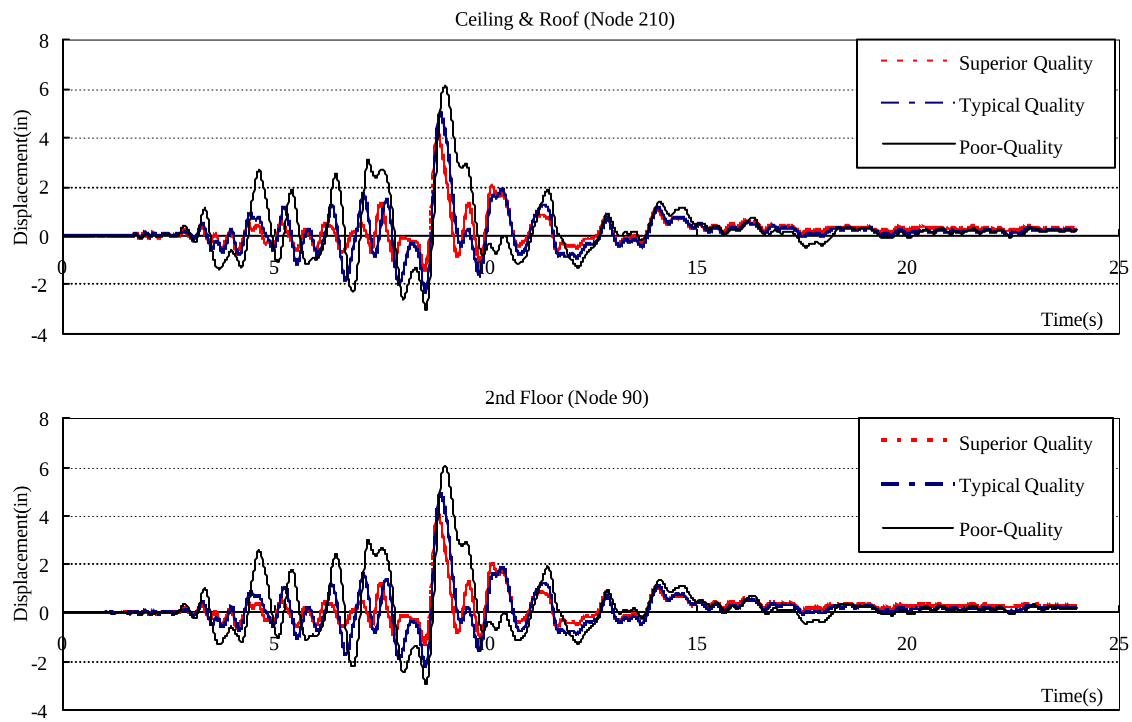


Figure 6.6 Displacement Time-Histories in the Y-Direction for Apartment Index Building Under Canoga Park Record, PGA = 0.50 g.

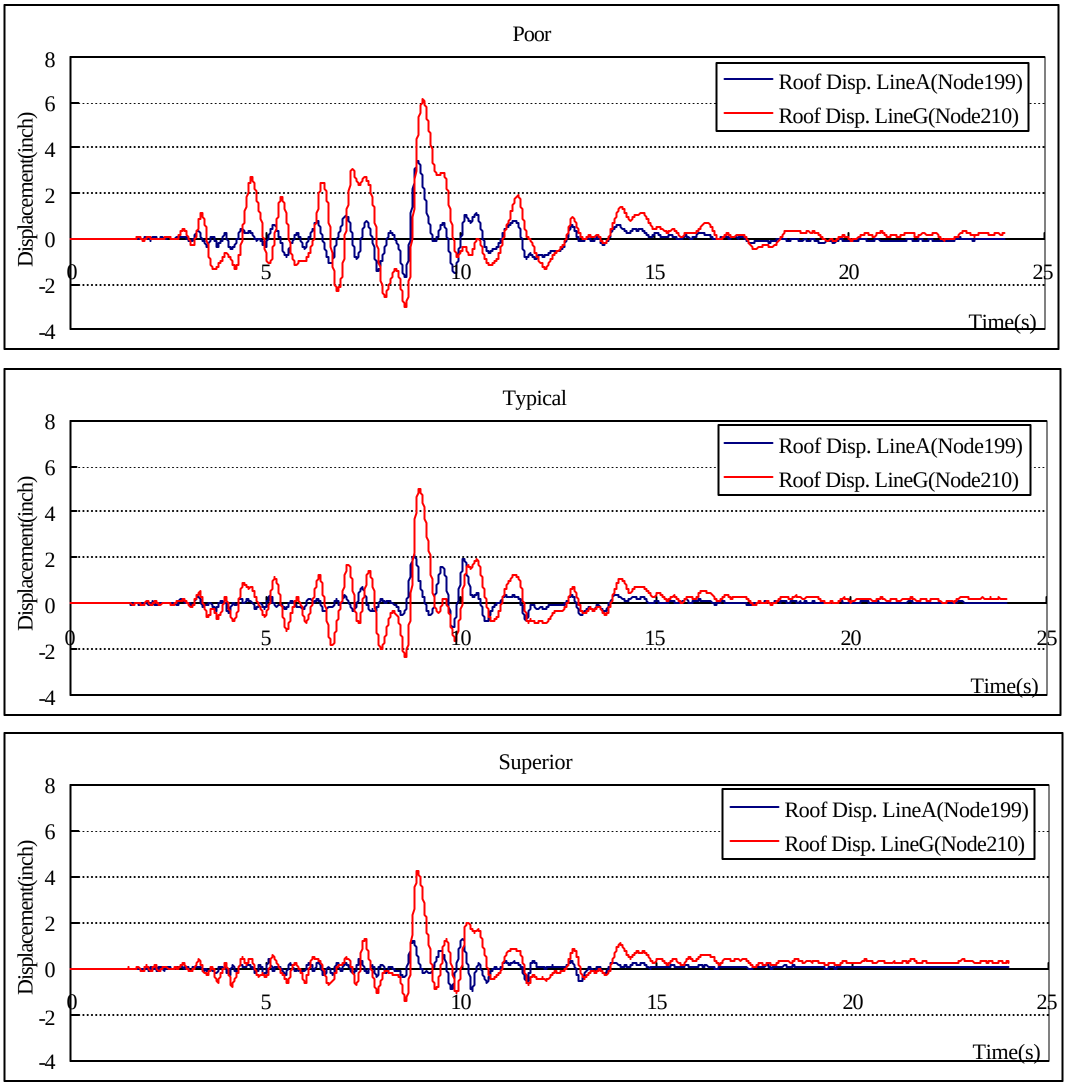


Figure 6.7 Displacement of Line A and Line G of Time-Histories in the Y-Direction for Apartment Index Building Under Canoga Park Record, PGA = 0.50 g.

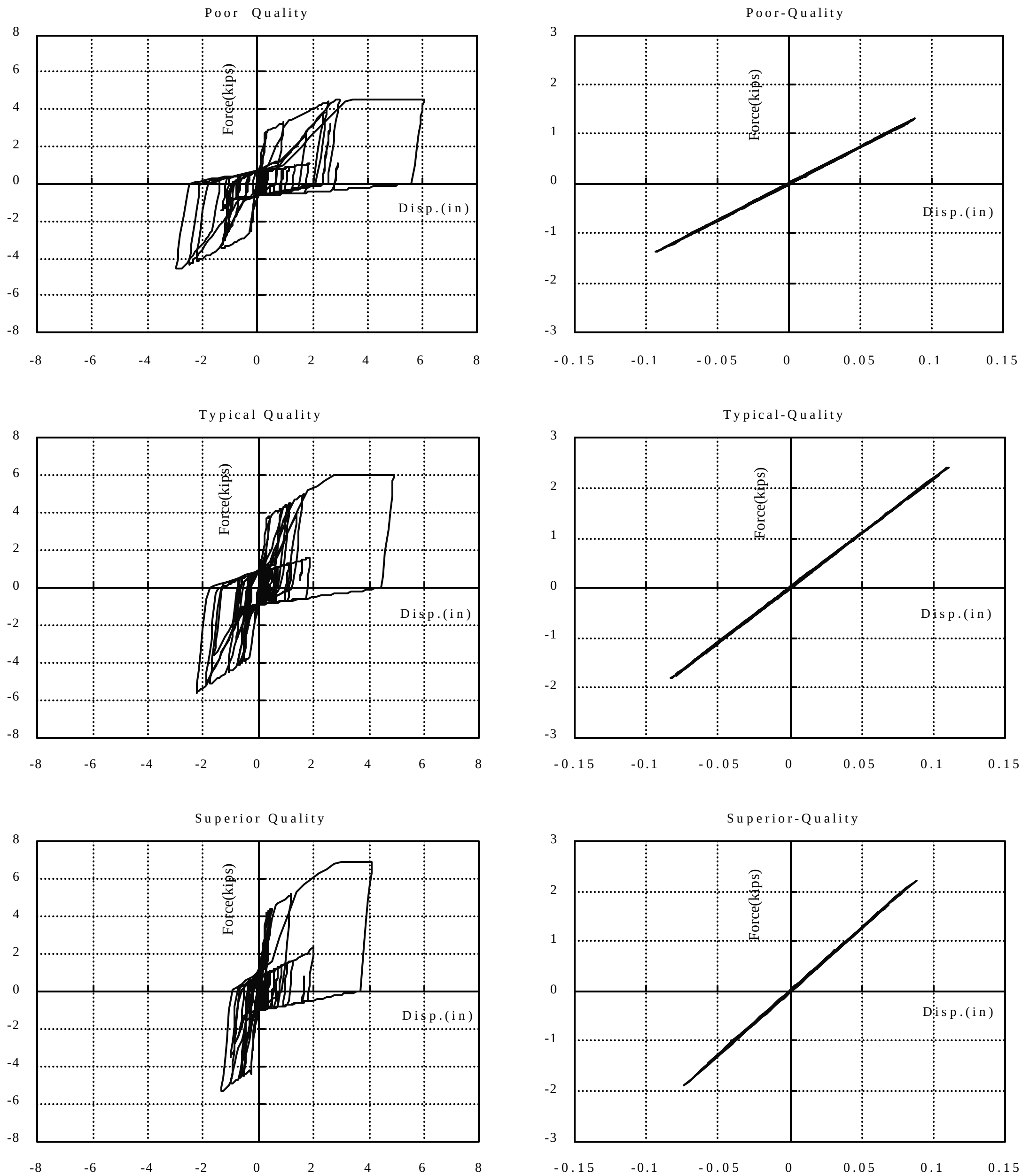


Figure 6.8 Hysteresis Loops of Walls Along Wall Line G (Element 542 on the left side and 642 on the right side) for Apartment Index Building Under Canoga Park Record, PGA = 0.50g.

6.12 Retrofit of Apartment Building: Retrofit Measure No.7 and No.8

The ground-level garage portion of the apartment index building was retrofitted with two different measures: retrofit measures No. 7 and 8 (note that the retrofit measure No. 6 was cancelled). The retrofit measure No. 7 consisted in installing five moment-resisting steel frames along the open front of the building, as shown in Figure 6.9 (Cobeen 2001). For the retrofit measure No. 8, on the other hand, new OSB shear walls were introduced in the interior of the building between wall lines 2 and 3, as shown in Figure 6.10 (Cobeen 2001).

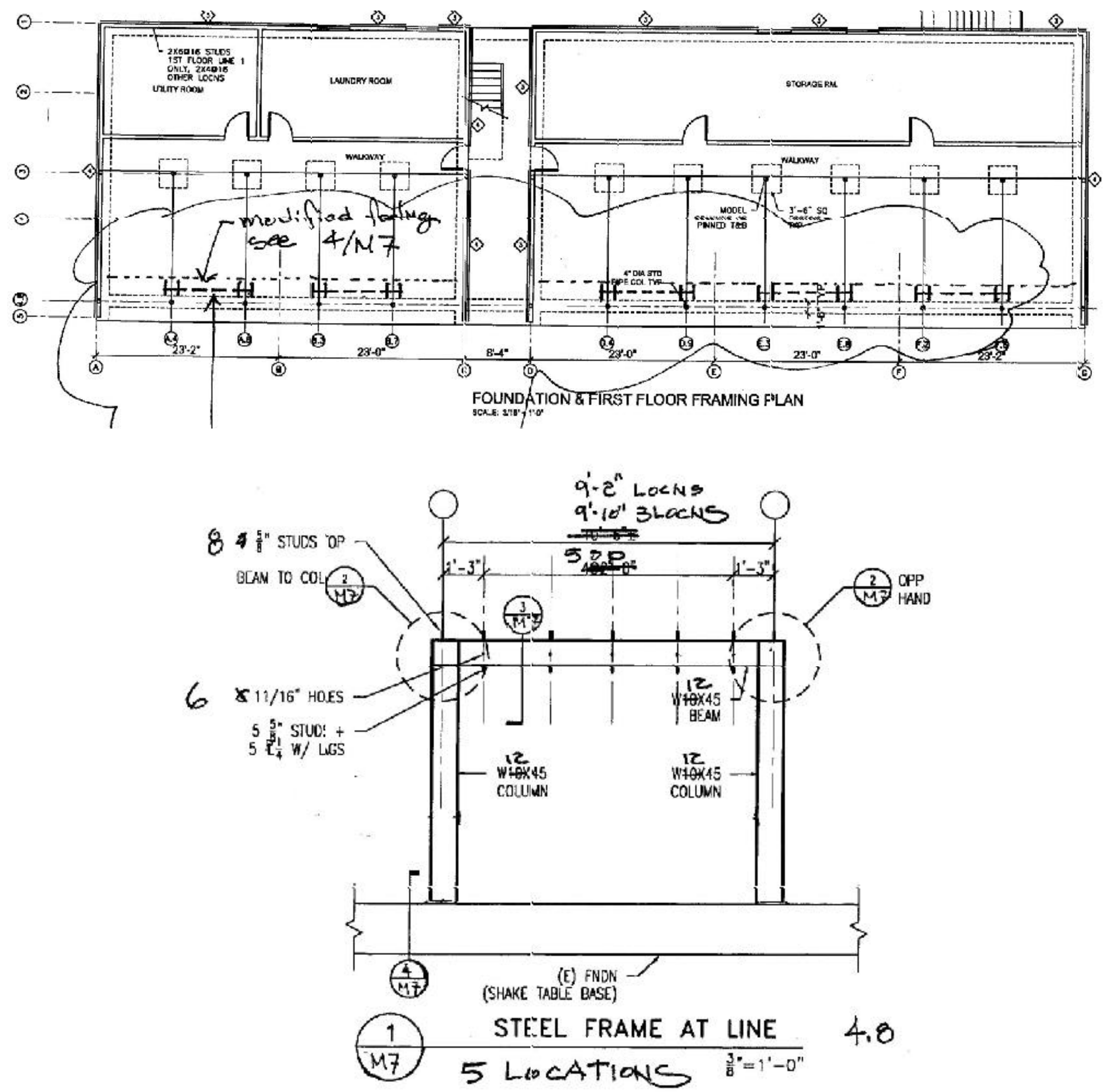


Figure 6.9 Locations and Details of Moment-Resisting Steel Frames Used in the Retrofit of Measure No. 7 of the Apartment Index Building (Cobeen 2001).

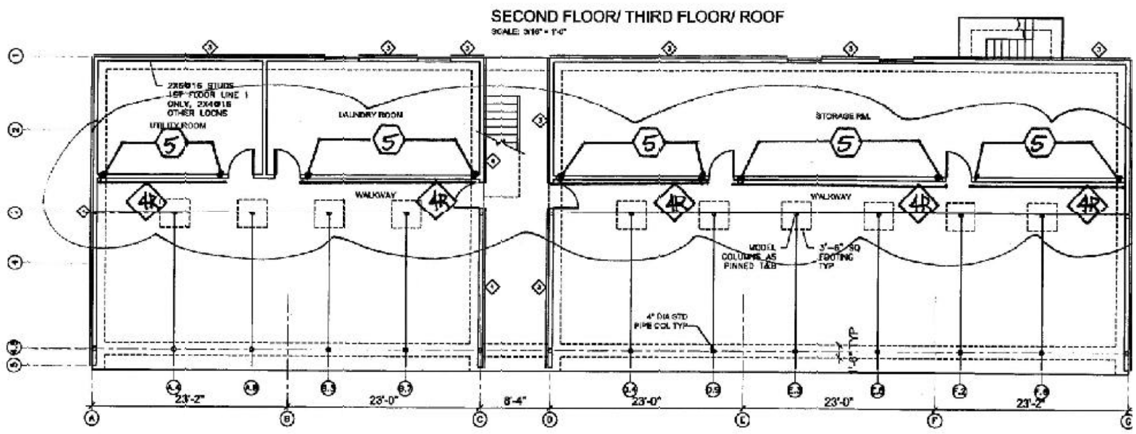


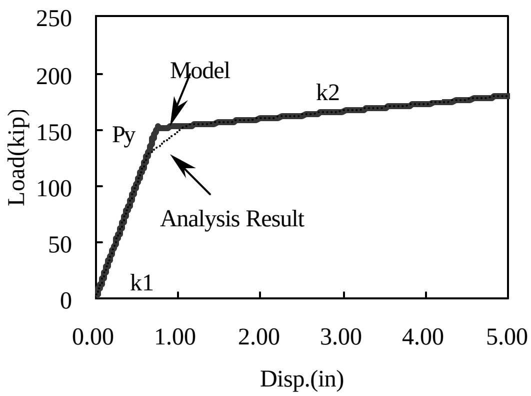
Figure 6.10 Locations of OSB Shear Walls Used in the Retrofit Measure No.8 of the Apartment Index Building (Cobeen 2001).

For the retrofit measure No. 7, the properties of the bi-linear hysteretic model used for each steel frame were computed based on the geometry and nominal properties of steel listed in Table 6.6. The resulting properties are shown in Figure 6.11. Each frame was converted into a single shear element exhibiting these properties.

For the retrofit measure No. 8, the details of the new shear walls were the same as those used for the retrofit measure No. 3, as shown in Table 4.13. Again, the properties of each shear element were obtained by adjusting the strength and stiffness values for the actual length of the full wall piers in each wall line, as shown in Table 6.7. The locations, orientations and numbering of the nodes and shear elements used in the retrofit measure No. 7 and 8 are shown in Figure 6.12 to 6.14.

Table 6.6 Geometries and Steel Properties for Moment-Resisting Steel Frames Used in the Retrofit Measure No. 7 of the Apartment Index Building.

Beam and Column Section	Area (in^2)	Depth (in)	I (in^4)	EI (kip/in^2)	E2	G	Plastic Hinge Length (in)
W12*45	13.2	12	394	29,000	0.02*EI	E1/2.6	0.9*Depth



Length	k1	k2	Py	Element
9'-2"	206.4	6.82	150.7	72
9'-10"	202.9	6.81	150.8	73

Figure 6.11 Properties of Steel Frame Used for Retrofit Measure No. 8 of Apartment Building Index Building.

Table 6.7 Parameters of Wayne Stewart Hysteresis Rule for Shear Elements of Retrofit Measure No. 8 of Apartment Index Building.

Wall Type	Story	Location	Direction	Shear Element No.	Fy (Kip)	Ko (Kip/in)	RF(r1)	Fu (Kip)	FI (Kip)	PTRI	PUNL	ALPHA	BETA
Vertical Shearwall OSB	1st	A23	X	72	5.91	18.90	0.077	9.59	1.42	-0.082	1.07	0.76	1.09
		B23	X	73	13.89	44.46	0.077	22.55	3.33	-0.082	1.07	0.76	1.09
		C23	X	74	7.99	25.57	0.077	12.97	1.92	-0.082	1.07	0.76	1.09
		D23,G23	X	75	6.95	22.23	0.077	11.28	1.67	-0.082	1.07	0.76	1.09
		E23,F23	X	76	14.93	47.80	0.077	24.24	3.58	-0.082	1.07	0.76	1.09

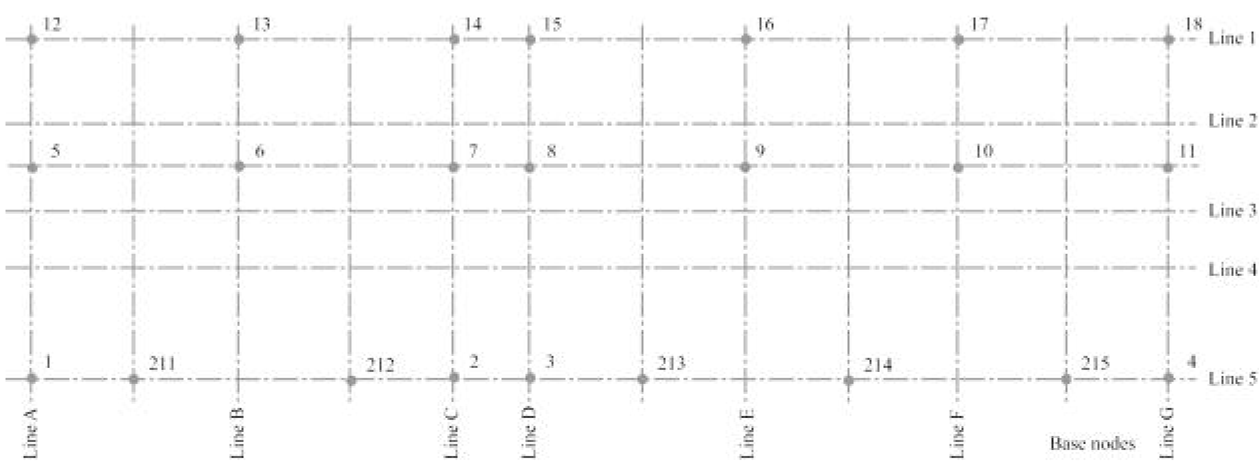


Figure 6.12 Node Numbering for Retrofit Measure No.7 of Apartment Index Building.

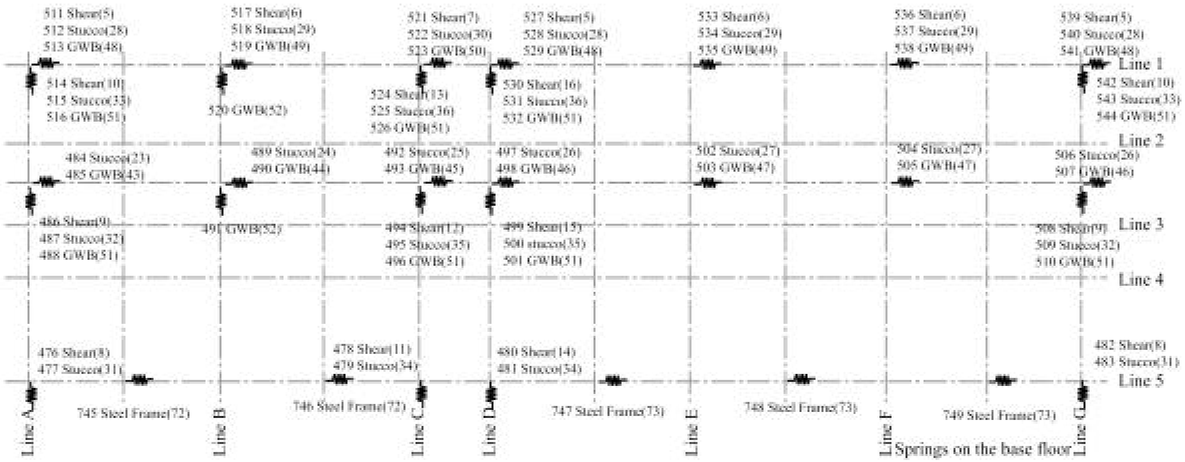


Figure 6.13 Locations, Orientation and Numbering of Shear Elements for Retrofit Measure No.7 of Apartment Index Building.

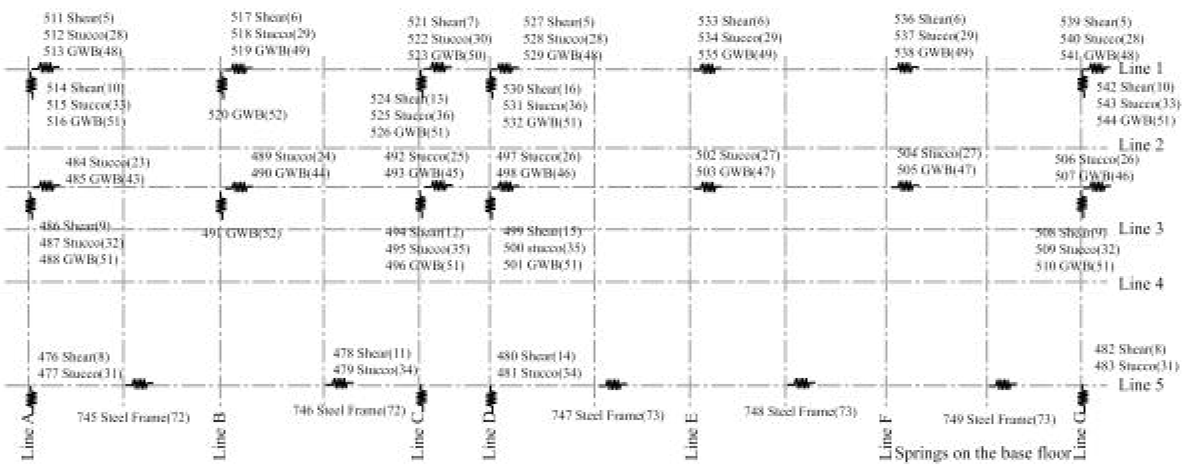


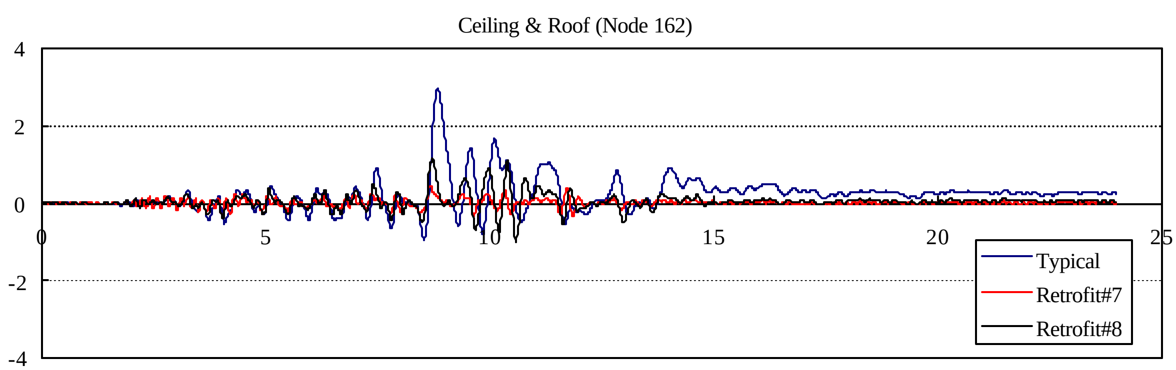
Figure 6.14 Locations, Orientation and Numbering of Shear Elements for retrofit measure No.8 of Apartment Index Building.

Both retrofit measures were applied to the typical-quality variant. For each retrofit measure, the building was excited parallel to its long side (x-axis direction) by the Canoga Park record of the 1994 Northridge Earthquake scale to a Peak Ground Acceleration (PGA) of 0.50 g. The data files for these two cases are included in the CD-ROM accompanying this report.

Table 6.8 compares the fundamental frequencies and maximum responses obtained with these two retrofit measures with that of the original typical-quality variant of the apartment index building. Figure 6.14 shows the time-history responses of node 30 on the second floor, and node 162 on the roof. It can be observed that each retrofit measure increased substantially the fundamental frequency of the original structure and enhanced significantly its seismic response. Particularly, the steel frames of retrofit measure No. 7 reinforce very well the large openings along the garage portion of the building.

Table 6.8 Fundamental Frequencies and Maximum Responses of the Retrofitted Apartment Index Building.

Construction Variant	Fundamental Frequency (Hz)	Maximum Response Node30 (in)	Maximum Response Node 162 (in)	Mode of Vibration
Typical-Quality	4.49	2.76	2.95	X-direction
Retrofit Measure No .7	5.97	0.15	0.45	X-direction
Retrofit Measure No. 8	4.69	0.84	1.13	X-direction



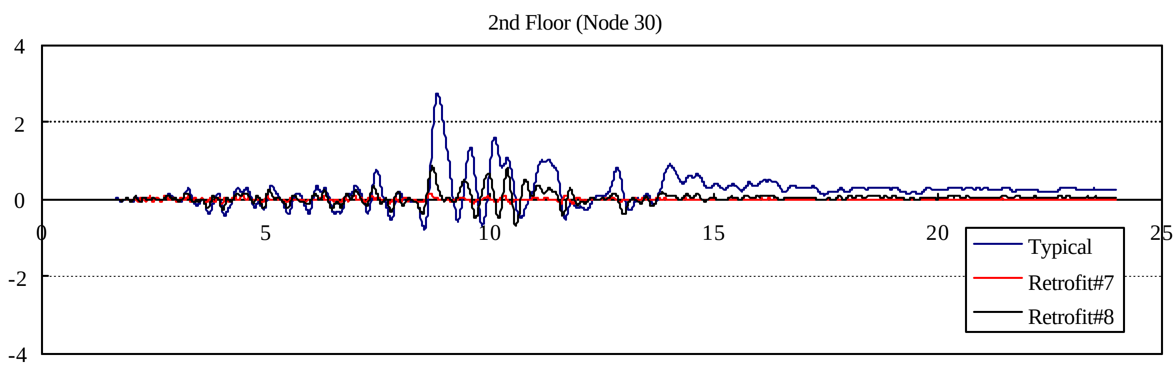


Figure 6.15 Displacement Time-Histories in the X-Direction for Apartment Index Building Under Canoga Park Record, PGA=0.5g.

7. CONCLUSIONS

The development of numerical models for performing deterministic nonlinear time-history analyses of four index woodframe buildings under various earthquake ground motions have been described in this report. These numerical models will be used for the analyses conducted by the Subcontractor of Task 4.1 (Improving Loss Estimation for Woodframe Building) of the CUREE-Caltech Woodframe project in order to improve the loss estimation methods for woodframe buildings.

The use of the pancake model to perform the three-dimensional nonlinear seismic analysis of complete woodframe buildings represents a simple and efficient approach to evaluate of global seismic response parameters. Nineteen different data files were developed for the three different construction variants for each of the four basic index buildings and for seven different retrofit measures of these index buildings. These 19 data files provide a set of hypothetical but realistic woodframe buildings that represent poor, typical and superior construction practices with various retrofit measures.

Based on the limited analysis examples performed in this project, the following conclusions can be drawn on the seismic response of the four index building considered.

<u>Index Building 1: Small House</u>

- The fundamental frequency computed based on the initial stiffness of the superior-quality variant of the small-house index building is much higher since the significantly more rigid concrete stem wall supports the weight of the floor.
- The cripple walls have a detrimental effect in the seismic response of the poor-quality and typical-quality variants of the small-house index building. For these two construction variants, practically all the displacements are the results of the cripple wall deformations. For the poor-quality variant, the maximum displacement exceeds the displacement at ultimate load of the cripple wall. The introduction of the concrete stem wall in the superior-quality variant reduces the displacements at the floor and ceiling levels by an order of magnitude.

- Retrofitting the cripple walls of the poor-quality and typical-quality construction variants with OSB sheathing reduces dramatically the displacements.

Index Building 2: Large-House

- The deformations are concentrated in the first floor shear walls in the three construction variants of the large-house index building.
- The introduction OSB panels below and above all window and door openings of the three construction variants of the large-house index building causes only a slight increase in natural frequency. The exterior Stucco is the main contributor to the lateral stiffness of the building.
- Retrofitting the typical-quality variant of the large-house index building by replacing all shear walls by new OSB walls 15/ 32 thick with an edge nail spacing of 3 inches increases substantially the fundamental frequency of the original structure and enhances significantly its seismic response. Replacing only the first floor shear walls, however, does not change substantially the seismic response of the structure.

Index Building 3: Townhouse

- The deformations are concentrated in the first floor shear walls in the three construction variants of the townhouse index building.
- Retrofitting the typical-quality construction variant of the townhouse index building by adding new shear walls in order to reduce the first floor drift is effective in reducing the seismic response of the structure.

Index Building 4: Apartment Building

- Because of the soft story introduced by the parking garages, the deformations are concentrated in the first floor walls in the three construction variants of the apartment index building.
- Retrofitting the typical-quality construction variant of the apartment index building by installing five moment-resisting steel frames along the open front of the building or by introducing new OSB shear walls in the interior of the building increase substantially the fundamental frequency computed based on the initial stiffness of the original structure and enhanced significantly its seismic response.

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(Appendices A-D of Isoda *et al.,* 2001, are omitted for brevity, as they duplicate the contents of Appendix B of Porter *et al.*, 2002, vol 2.)

Appendix D. ABV Methodology

The following article is reprinted with permission from *Earthquake Spectra,* 17 (2), May 2001, Oakland, CA: Earthquake Engineering Research Institute, 291-312

Assembly-Based Vulnerability of Buildings and Its Use in Performance Evaluation

Keith A. Porter, M.EERI, **Anne S. Kiremidjian,** M.EERI, **and Jeremiah S. LeGrue**

Assembly-based vulnerability (ABV) is a framework for evaluating the seismic vulnerability and performance of buildings on a building-specific basis. It utilizes the damage to individual building components and accounts for the building's seismic setting, structural and nonstructural design and use. A simulation approach to implementing ABV first applies a ground motion time history to a structural model to determine structural response. The response is applied to assembly fragility functions to simulate damage to each structural and nonstructural element in the building, and to its contents. Probabilistic construction cost estimation and scheduling are used to estimate repair cost and loss-of-use duration as random variables. It also provides a framework for accumulating post-earthquake damage observations in a statistically systematic and consistent manner. The framework and simulation approach are novel in that they are fully probabilistic, address damage at a highly detailed and building-specific level, and do not rely extensively on expert opinion. ABV is illustrated using an example pre-Northridge welded-steel-moment-frame office building.

INTRODUCTION

Seismic vulnerability functions, also called motion-damage relationships, are used to estimate earthquake losses and to aid in making seismic risk-management decisions. Figure 1 schematically illustrates a probabilistic vulnerability function for a single building. The horizontal axis shows the spectral acceleration to which the building is exposed, while the vertical axis shows cost as a fraction of building value (often called the damage factor, denoted by Y). The figure shows that at any level of spectral acceleration, denoted by S_a, the damage factor Y is uncertain, with an associated probability distribution that depends on S_a.

One can categorize current methodologies to create motion-damage relationships in two groups: techniques based on structure type—a broad category into which a particular building falls—and techniques based on a detailed structure analysis of the particular building. Among the most familiar and popularly used category-based methodologies are ATC 13 (Applied Technology Council, 1985), and HAZUS (National Institute of Building Sciences, 1997). These approaches characterize a building by its lateral force resisting system and height, and apply generic pre-established vulnerability functions to determine repair cost and loss of use duration. These methods offer simplicity and general applicability, and allow for probabilistic estimation of earthquake-related losses on a regional level. But because they rely on a limited number of structure types, loss estimation for a particular building is problematic: one cannot account for the building's unique structural and nonstructural design.

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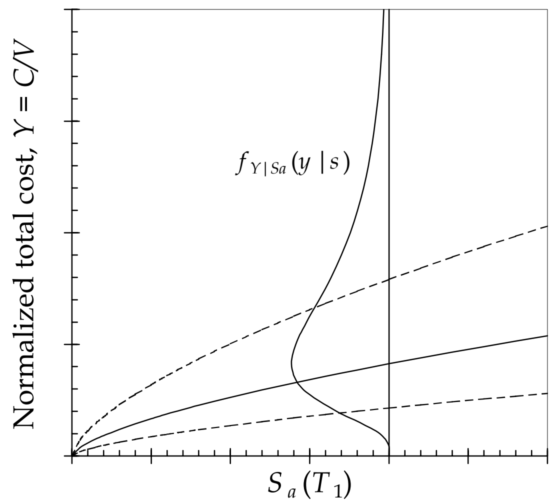

Figure 1. A seismic vulnerability function, in schematic form. Three curves are shown: the mean total earthquake loss as a fraction of replacement cost, and two dashed lines representing ± 1 standard deviation. The figure shows that loss at any particular level of spectral acceleration, loss is uncertain, and has an associated probability distribution, $f_{Y|Sa}(y|s)$.

Several studies have proposed to estimate damage and repair cost based on the structural and nonstructural design of an individual building. Scholl (1980) proposed a methodology to create building-specific seismic vulnerability functions. His approach was further developed by Kustu et al. (1982) and Kustu (1986). In it, structural analysis using response spectra is used to calculate peak structural response in terms of floor drift ratios and peak floor accelerations. The structural response is input to component damage functions, that is, relationships between structure response and damage state of typical building components. Thus one determines the damage state of building components on a floor-by-floor basis.

Using statistics of mean repair cost for each damage state, the analyst can estimate mean total repair cost by component category, summing to determine total cost. The approach is largely deterministic, and a variety of building components are lumped together in the component damage functions. For example, all mechanical, electrical, and plumbing components are reflected in just one or two curves. Because of this, it is impossible to account for differences in performance between alternative component types or installation conditions. Because one cannot distinguish between performance before and after a seismic rehabilitation measure such as anchoring equipment, the benefits of the rehabilitation cannot be evaluated, making cost-benefit analysis impossible. On the more fundamental level of scientific verification, since components are highly aggregated, it can be highly problematic to create or perform laboratory tests to check the aggregate component damage functions.

ASSEMBLY-BASED VULNERABILITY

This paper summarizes a study (Porter, 2000) that developed a new method of calculating seismic vulnerability for individual buildings. The new method, called assembly-based vulnerability (ABV), extends the Scholl (1980), Kustu et al. (1982) and Kustu (1986) technique. The new approach allows for probabilistic performance evaluation and considers building assemblies at a greater level of detail than do these earlier approaches. The technique employs methods used since the early 1980s on probabilistic risk assessments of nuclear power plants.

A simulation approach to implementing ABV is illustrated in Figure 2. Its steps are summarized here and detailed below. Each simulation proceeds as follows: first, one determines the building location, site conditions, and design details, including structural and nonstructural components. Then one selects or generates an acceleration time history appropriate to the building site. Next, a structural analysis is performed to determine the building's peak structural response to that input ground motion. Various parameters of the structural response are then recorded: peak floor accelerations, peak transient drifts, peak member forces, and so on.

Fourth, for each assembly in the building, the appropriate structural response parameter is input to one or more assembly fragility functions to determine the probability that the assembly will be damaged and require repair or replacement. By a method that will be described below, this probability is used to simulate the damage state of each assembly in the building. Fifth, using a probability distribution on the unit cost to repair each assembly, and another on the time required to repair each assembly, one simulates the cost and time to repair all the damaged assemblies. The costs are added up to produce a simulation of the total repair cost. The durations are used in a construction-scheduling procedure to produce a simulation of the loss-of-use duration, which is then used to estimate the loss-of-use cost.

Thus is completed one simulation of shaking, response, damage, repair, and loss. To create a complete seismic vulnerability function, the procedure is repeated many times for each of many levels of ground shaking intensity that the building might experience. Details of these steps are now presented.

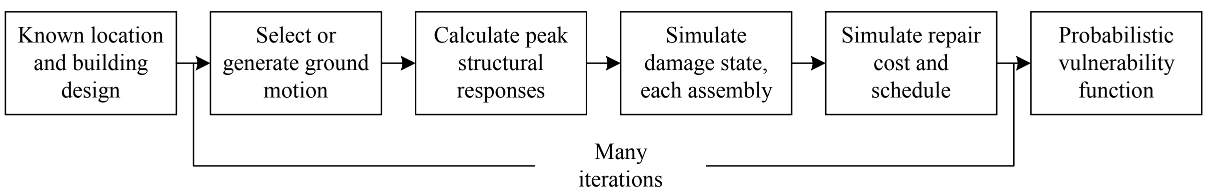

Figure 2. Steps of the ABV methodology.

BUILDING DESIGN

First, the building's location, site conditions, and detailed design are determined. For existing buildings, this includes an examination of actual installation conditions, particular with regard to seismic anchorage or bracing of nonstructural components. A structural model is then created, using techniques familiar to structural engineers. In addition, all damageable structural and nonstructural components in a building must be inventoried using a formal categorization system (or taxonomy) of assembly types, including all damageable structural, architectural, mechanical, electrical, and plumbing assemblies. This step need not be excessively burdensome if assemblies are defined at a moderately aggregated level. There are several advantages of using a standard taxonomy for building assemblies, and of using only moderately aggregated assemblies:

1. A standard taxonomy establishes a common language that researchers and designers can use to compile, exchange, understand, and use damage and loss data.
2. If the taxonomy aligns well with the category systems used in construction-cost estimation, then repair-cost estimation can take advantage of published cost

manuals. These manuals reflect the results of extensive, ongoing surveys of the construction industry by professional cost estimators.

3. By referring to a complete taxonomy, researchers can be sure that they are not ignoring important damageable components when they estimate losses to a particular building.
4. Assemblies defined at a moderately aggregated level such as gypsum wallboard partitions can be readily tested in a laboratory for seismic resistance. At a greater aggregation, such as “nonstructural partitions,” laboratory testing becomes difficult.

The categorization systems most familiar to construction cost estimators in the United States are those of the RS Means Co. (1997a and 1997b). The assembly category system of the RS Means Co. (1997b), which itself is an extension of the standard UniFormat system (Construction Specifications Institute, 1998), is used here because it represents a reasonable balance between the effort required to create an inventory, and the need to distinguish between the performance of assemblies with similar function but markedly different seismic vulnerability. Assemblies at about this level of aggregation are commonly tested in laboratories to determine their seismic performance.

The assembly taxonomy system of RS Means Co. (1997b) uses three levels of detail to categorize assemblies: division and subdivision, major classification, and line. This system is extended here to account for details of design and installation that are relevant to earthquake damage. First, an additional taxonomic division, entitled *condition*, is added to characterize adequacy of installation for seismic resistance, for example, to indicate seismic anchorage or bracing conditions. Also, new lines are defined to account for problems idiosyncratic of earthquake damage and repair. For example, pre-Northridge welded-steel moment frame (WSMF) connections are separated from beams and columns to reflect their unusual seismic fragility and repair requirements.

GROUND-MOTION SELECTION

In the next stage, ground acceleration time histories are selected for the site. To create a complete vulnerability function, time histories are selected and scaled to cover a range of values of spectral acceleration at the building’s fundamental period, denoted by $S_a(T_1)$. The S_a levels range from a small value s_{min} such as 0.05g, through a large value s_{max} that building might experience. Several such recordings must be used for each $S_a(T_1)$ value, in order to capture the variability of detailed time histories with the same spectral acceleration. Ground motions can be scaled within reasonable bounds to produce the required incremental steps in spectral acceleration, but should reflect approximately the proper magnitude, faulting mechanism, distance, and site soil conditions.

Ground motion records can be historic or simulated. They can be created for example from real records using an autoregressive moving average (ARMA) model. Such a simulation approach may be necessary because of the large number of iterations required to create a robust probability distribution on loss. For a discussion of ARMA techniques, see Box et al. (1994), Polhemus et al. (1981), and Conte et al. (1992).

A nonstationary ARMA model was used in the illustration of the present study because it has been successfully used in the past and because it generates ground motion time histories that have amplitude and frequency content that vary over the duration of the record to match the changing characteristics of the real record (Singhal et al. 1997).

STRUCTURAL ANALYSIS

Using the ground motion record prepared in the previous step, the time-history response of the structure is calculated using a nonlinear dynamic structural analysis. Key structural responses are recorded. The response variables of interest depend on the assemblies present in the building. They can include peak values such as peak transient drifts or peak accelerations, or they can reflect an accumulated values such as hysteretic energy dissipated by a particular component. To account for the uncertainty of peak structural response given a particular ground motion, building dimensions, member stiffness, and masses can be treated as random variables and samples generated for each iteration.

BUILDING ASSEMBLY DAMAGE

The objective of the next step is to generate the damage state of every assembly in the building. The building and all of its assemblies are assumed to be undamaged before the earthquake. The peak structural responses collected from the previous step are then input to assembly damageability relationships to simulate the damage state of each assembly. In the present treatment, damage to each assembly is considered to be dependent on only one structural response parameter. It may be that damage to some assemblies can be better predicted by considering two or more response parameters. For simplicity, however, this possibility is not examined here.

More than one possible damage state can be defined for each assembly type. Let N denote the number of possible damage states for a particular assembly type. The N possible damage states must be defined to be mutually exclusive and, with the addition of the undamaged state, collectively exhaustive. Let D represent a discrete random variable to indicate damage state, that is, $D \in \{0, 1, \ldots N\}$, and let d denote a particular value that the random variable takes on for a particular assembly.

The damage state of a particular assembly will depend on the structural response to which it is subjected. Let Z denote the (uncertain) structural response to which a particular assembly is subjected, and let z denote a particular value of Z. As a discrete random variable that depends on Z, D has a conditional probability mass function, denoted by $p_{D|Z}(d|z)$, and a conditional cumulative distribution function, denoted by $F_{D|Z}(d|z)$ (A glossary is provided at the end of this paper that summarizes these and other terms.)

Let the capacity of an assembly type to resist a particular damage state d be represented by an uncertain variable X_d. If the assembly is subjected to structural response $z < X_d$, it does not enter that damage state. If on the other hand $z > X_d$, then the assembly reaches or exceeds damage state d. Because it is a random variable, the capacity X_d has an associated cumulative probability distribution, denoted by $F_{Xd}(x)$. Observe then that the cumulative distribution evaluated at the level of structural response z gives the probability that a particular assembly will reach or exceed damage state d, as shown in Equation 1.

$$P[D \geq d| Z = z] = P[X_d < z] = F_{Xd}(z) \tag{1}$$

where

$P[A]$ = the probability that A is true

$P[A \mid B]$ = the probability that A is true, given condition B

$D \in \{0, 1, \ldots N\}$, represents damage state, where $D = 0$ refers to the undamaged state;

N = number of damage states defined for the assembly. $N = 0$ implies that the assembly is assumed to be rugged, not damageable in an earthquake;

X_d represents the assembly's capacity to resist damage state d; and

$F_{Xd}(z)$ represents the cumulative probability distribution function of X_d evaluated at z.

After an earthquake, a particular assembly is either undamaged ($D = 0$) or in one of several damage states 1, 2, … N. As a simplifying assumption, damage states are assumed to be *progressive*, that is, an assembly passes through damage state d to reach damage state d+1. This appears to be reasonable for the structural and architectural assemblies studied here. If a particular assembly has two or more possible damage states, and each damage state has an associated random capacity X_1, X_2, … X_N, then the probability that the assembly is in or exceeds a particular damage state decreases with each higher damage state, as shown in Equation 2.

$$F_{Xj}(x) \leq F_{Xi}(x) \qquad \text{for } x > 0,\ 1 \leq i < j \leq N \tag{2}$$

Equations 1 and 2 imply equation 3, which provides the probability mass function for the damage state of an assembly, given the structural response to which the assembly is subjected. This equation says that the probability that an assembly is in damage state d equals the probability that it is damage state d or higher, minus the probability that it is in damage state d+1 or higher.

$$\begin{aligned} p_{D|Z}(d \mid z) &= P\left[D = d \mid Z = z\right] = P\left[D \geq d \mid Z = z\right] - P\left[D \geq d+1 \mid Z = z\right] \\ &= F_{Xd}(x) - F_{Xd+1}(x) \end{aligned} \tag{3}$$

By considering each damage state in turn, a conditional probability mass function and conditional cumulative probability distribution on damage state D can be created (Equations 4 and 5).

$$p_{D|Z}(d \mid z) = \begin{cases} 1 - F_{X1}(z) & d = 0 \\ F_{Xd}(z) - F_{Xd+1}(z) & 0 < d < N \\ F_{Xd}(z) & d = N \end{cases} \tag{4}$$

$$F_{D|Z}(d \mid z) = \sum_{\delta=0}^{d} p_{D|Z}(\delta \mid z) \qquad 0 \leq \mathrm{d} \leq \mathrm{N} \tag{5}$$

Thus, one compiles the capacity distributions for each damageable assembly in the building, and uses these with the results of the structural analysis to create the conditional cumulative probability distribution for the damage state of each assembly. In order to simulate the damage state for an assembly, a sample value u is generated from the uniform (0,1) distribution. (That is, a sample value of the random variable U is drawn, where $0 < U \leq 1$, and every possible value of U has equal probability). The damage state d for each assembly is then evaluated from the inverse cumulative distribution of damage, given by Equation 6. The number of assemblies of each type in each damage state can then be added up. Let $N_{j,d}$ represent the number of assemblies of type j that are in damage state d in a particular simulation.

$$d = F_{D|Z}^{-1}(u) \tag{6}$$

Note that damage states must be defined in terms of particular repair tasks required to restore the assembly to an undamaged state. As used here, each damage state refers to an observable and unambiguously defined condition of the individual assemblies in the building, not using building-wide macroscopic damage states such as minor, moderate, etc., which are commonly used by category-based approaches such as ATC 13 (Applied Technology Council, 1985) and HAZUS (National Institute of Building Sciences, 1997). No building-wide damage state or damage index is defined in the ABV framework.

The assembly capacity distributions of Equation 1 can be created by a variety of means. Empirically based capacity distributions can be created from laboratory experiments or from earthquake experience. For example, Swan et al. (1998) describe a method to derive a capacity distribution from earthquake experience data. That study focuses on component functionality, that is, whether a particular piece of equipment is functional or not. However, the same approach can be used to derive capacity distributions that refer to other types of physical damage such as whether a window is cracked or fallen out. Where inadequate laboratory or earthquake experience data exist, under certain conditions one can create theoretical capacity distributions using reliability methods.

If data are inadequate to create an empirical or theoretical capacity distribution for an assembly, a judgment-based distribution can be used. The HAZUS methodology, for example, relies extensively on judgment to create aggregate component fragility functions from the empirical detailed fragility data that were available to the investigators (National Institute of Building Sciences, 1997). It was desired to avoid reliance on expert opinion for the present study, because expert opinion is often perceived by decision-makers to weaken the credibility of the overall analysis.

However, where judgment-based capacity distributions are necessary, the quality of expert opinion can be maximized through careful means of eliciting judgment. Tversky et al. (1974) discuss some of the potential problems—biases of judgment—that arise when eliciting judgments of probability. Spetzler et al. (1972) detail a methodology to interview experts to encode probability beliefs, with due attention to minimizing the effect of such biases.

REPAIR COST

The purpose of the next step is to generate the total repair cost for each simulation, based on the damage states of all the damageable assemblies in the building. Let $C_{j,d}$ denote the uncertain cost to restore one unit of assembly type j from damage state d to an undamaged condition. It reflects the direct cost to the owner including all materials, equipment, labor. Unit repair costs are assumed to be random variables with characteristic cumulative probability distribution $F_{Cj,d}(c)$, where c is a given value of unit cost. One must compile the cost distribution for each damage state of each damageable assembly in the building.

The repair cost $C_{j,d,}$ is simulated by generating a sample u from the uniform (0,1) distribution and applying the inverse method. That is, in the simulation, the unit cost is taken as the inverse cumulative distribution of $C_{j,d}$ evaluated at u. The total repair cost for the building is the sum of unit repair costs times the number of damaged assemblies (Equation 7).

$$C_R = \sum_j \sum_d F_{Cj,d}^{-1}(u) N_{j,d} \qquad (7)$$

where

$C_{j,d}$ = the uncertain cost to repair an assembly of type j from damage state d

$F_{Cj,d}^{-1}(u)$ = the value of the unit repair cost $C_{j,d}$ with non-exceedance probability u.

C_R = total repair cost

$N_{j,d}$ = number of assemblies of type j in damage state d

A refinement of this approach is to separate the costs not directly attributable to the repair of particular assemblies, such as contractor overhead and profit, mobilization and demobilization, etc., and to calculate these separately, rather than including them in $C_{j,d}$.

LOSS OF USE

Earthquake losses accrue from loss of use as well as from direct damage, so it is necessary to estimate the time to repair the building. If income derives from the operation of parts of the building, such as from rent from apartments, office suites, or floors, then it is necessary to estimate the repair duration from each part. These parts are referred to here as the operational units of the building.

If each operational unit in the building can be described as requiring a set of critical structural, architectural, mechanical, electrical and plumbing features to be functional, then the time to restore the damaged critical components can be used to estimate loss of use cost.

One can estimate the time to restore critical components using standard scheduling procedures. A schedule can be visualized with a Gantt chart, which depicts tasks as horizontal bars whose length indicates the duration of each task. Vertical and horizontal lines connect the bars; these lines indicate the order in which tasks must be completed. To estimate loss of use duration then, one must determine which tasks must be performed, the order in which they can be performed, and the duration of each task.

Duration of one repair task

Each task in the schedule consists of repairing all similar assemblies in one operational unit of the building. For example, one task might be to repair all broken windows in a particular office suite. The time required for a standard construction crew to restore one unit of assembly type j from damage state d to the undamaged state is denoted $U_{j,d}$. Unit repair durations are assumed to be random variables with characteristic cumulative probability distribution $F_{Uj,d}(\nu)$, where ν is a particular value of $U_{j,d}$.

During a simulation, the time to repair assemblies of type j from damage state d can be generated by generating a sample u from the uniform (0,1) distribution and applying the inverse method, as shown in Equation 8. The duration of repairs to all assemblies of type j in damage state d located in operational unit m is given by Equation 9.

$$U_{j,d} = F_{Uj,d}^{-1}(u) \tag{8}$$

$$R_{j,d,m} = \frac{N_{j,d,m} U_{j,d}}{wE_j} \tag{9}$$

where

$R_{j,d,m}$ = workdays to restore all instances of assembly type j located in operational unit m from damage state d.

$N_{j,d,m}$ = number of instances of assembly type j located in operational unit m that are in damage state d. This is counted up from the damage simulation data, just as $N_{j,d}$ was. In fact, $\sum_{m} N_{j,d,m} = N_{j,d}$.

$U_{j,d}$ = time for one crew to restore one instance of assembly type j from damage state d to an undamaged state, measured in hours. Mean values are published in cost manuals, and variances can be estimated.

w = number of working hours per workday.

E_j = number of crews available for restoring assembly type j. The type of work and construction practice typically determines the crew size.

Duration of repairs to an operational unit and loss of use cost

Components are restored in a logical order that is dictated by construction practice, facility layout, tenant needs, and the construction contractor's labor and subcontractor availability. Scheduling is therefore idiosyncratic to a repair contract, and modeling for the generic case is problematic. However, several simplifying assumptions can be made to approximate the actual schedule:

- Within an operational unit, crews working on the same repair task work in parallel. Different repair tasks are performed in series.

- Repairs are performed in an order that follows the numbering of MasterFormat divisions, following standard procedures for new construction. For example, structural components are repaired before nonstructural finishes.

- Constraints due to tenant requirements are neglected while critical components remain unrepaired, on the assumption that tenants cannot occupy the facility until critical repairs are completed.

- In an operational area, one trade operates at a time. When that trade completes its work, the next trade is free to begin, once it completes its work elsewhere on site. If the next trade is not currently on site, a change-of-trade delay occurs.

- The duration of a change-of-trade delay varies depending on the size and complexity of the work and on labor and subcontractor availability, which may in turn depend on local economic factors. Bounding cases can be assumed. The slow-repair case has long change-of-trade delays of days or weeks; the fast-repair case can have short change-of-trade delays of one or two days.

- Repairs to different operational units can begin simultaneously if sufficient contractor labor is available. Alternatively, a contractor can concentrate on one operational unit until work for its trade is completed, then move on to the next operational unit where repairs appropriate for the contractor's trade await. These situations can be included in the bounding cases: the first, fast repair; the second, slow.

With these assumptions in mind, it is possible to estimate the time required to repair a single operational unit. Let

R^*_m = time to repair operational unit m, measured in workdays from the date on which repair work is begun in the facility.

$R_{j,d,m}$ = time to restore all instance of assembly type j located in operational unit m from damage state d to an undamaged state, measured in days, from Equation 10.

R_T = change-of-trade delay. Can be assumed based on bounding cases.

$R_{T0,m}$ = initial delay before first task in operational unit m. Can be assumed based on bounding cases.

n_m = number of trade changes involved in the repair of operational unit m. Determined from the damage simulation.

Equation 10 estimates the time required to repair damage in operational unit m.

$$R^*_m = \sum_j R_{j,d,m} + n_m R_T + R_{T0,m} \tag{10}$$

Loss-of-use cost is often a direct function of loss-of-use duration, the simplest example being lost income on a rental property (Equation 11).

$$C_U = \sum_m R_m U_m \tag{11}$$

where

C_U = total loss-of-use cost

R_m = time to repair operational unit m, measured in calendar days from the earthquake, accounting for R^*_m plus weekends and time between the earthquake and the date on which work is begun. R_m is bounded below by repair duration for building-service equipment and repair duration for common access areas.

U_m = daily rental income from operational unit m

Peculiarities of individual lease arrangements are not reflected in this simple relationship, e.g., voiding of a lease if the building is unavailable for an extended period of time, relocation costs, or costs reflecting higher lease at the temporary location of the tenant. For a specific building these costs are understood relatively well and can be easily included in the model.

TOTAL COST

Total cost, denoted by C, consists of direct repair cost (C_R, from Equation 7) and loss-of-use cost, denoted by C_U, from Equation 11. Other indirect costs and benefits are not captured in this equation, for example, changes in building value associated with perceptions of the safety or the building, code-compliance requirements triggered by repairs, or market effects such as demand surge. (Demand surge refers to the increase in repair costs sometimes associated with catastrophic earthquakes and hurricanes.)

$$C = C_R + C_U \tag{12}$$

COMPILING A VULNERABILITY FUNCTION

As noted above, each simulation produces a single value of C. Numerous simulations for each value of S_a will produce a range of samples of C. From these samples, one can calculate the mean value of cost, its standard deviation, and shape of its distribution. By repeating the process over a wide range of spectral accelerations, and performing regression analysis on the resulting data, one produces a probabilistic seismic vulnerability function for a particular building.

This vulnerability function is similar to those produced by category-based methodologies such as ATC 13 (Applied Technology Council, 1985) and HAZUS (National Institute of Building Sciences, 1997), with some important differences. First, the ABV vulnerability function accounts for the building's unique details of structural design because it uses structural analysis of that particular building. Second, it accounts for the building's unique architectural, mechanical, electrical, and plumbing features and details of their installation, rather than relying on the broad assumptions and judgments that are necessary when applying category-based approaches to particular buildings. Third, details of the causes of cost are available to identify which assemblies or portions of the building are contributing most strongly to overall cost. This allows a designer or analyst to quantify the costs and benefits of changing the components or their installation conditions, that is, to assess the value of seismic rehabilitation measures.

These benefits come at a significant expense of labor: an ABV analysis requires the compilation of numerous ground motion recordings, the creation of an analytical model of the structure, and an inventory of the building's assemblies. For assemblies whose capacity and repair cost distributions are not already known, these must also be created, which is potentially the most time-consuming aspect of the methodology. Once these items are available, however, the actual computation is fairly straightforward and can be readily automated, and the assembly fragilities can be re-used in later studies.

RANDOMNESS AND UNCERTAINTY

The ABV approach results in an explicit, defensible estimate of the uncertainty in the seismic vulnerability of a building. This is important, because uncertainty is a key feature of seismic risk management decisions. If one had perfect knowledge of when earthquakes occur and exactly how much damage they do, earthquake loss management would be as simple as cost-benefit analysis: choose the seismic design, strengthening scheme, or other measure that results in the greatest present value of the building. But imperfect knowledge is the rule in each aspect of earthquake loss estimation: when and how strong an earthquake will occur; the response of the structure to the earthquake; the consequent damage; and the costs to repair the damage.

Because imperfect information causes such great uncertainty in the timing and amount of future losses, and because those losses can represent a large fraction of a building owner's total wealth, cost-benefit analysis (which assumes risk neutrality on the decision-maker's part) is an inappropriate decision-making approach. To understand the amount of resulting uncertainty and its sources is to begin to manage it. If one can quantify uncertainty on loss as well as its mean value, one can use sophisticated decision-making methodologies such as decision analysis, which explicitly consider uncertainty and the decision-maker's risk attitude. A decision-analysis approach to making building-specific seismic risk management decisions is described in Porter (2000).

USING ABV TO CHECK PERFORMANCE-BASED DESIGN OBJECTIVES

The damage-estimation technique of ABV also provides the data needed to verify numerical performance-based design objectives. Current code design methods safeguard life-safety and serviceability by prescribing strength and stiffness requirements for the structural system, with limited focus on nonstructural building aspects. By contrast, Vision 2000 (Structural Engineers Association of California, 1995) and FEMA 273 (Applied Technology Council, 1997) attempt to establish a performance-based design (PBD) philosophy whose goal is to satisfying broader, predictable seismic performance objectives.

FEMA 273 associates four performance levels—operational, immediately occupiable, life safe, and collapse prevention—with varying earthquake hazard levels: 50% exceedance probability in 50 years, 20% in 50 years, 10% in 50 years, and 2% in 50 years. For each performance level, performance objectives are defined in detail by structural element and other building components. Component performance is expressed generally in qualitative terms such as "isolated dislocations," "minor cracking," or "generally operational." These terms, while not directly useful in an engineering calculation, can be associated with numerical values. Table 1 shows a sample translation of qualitative terms to numbers. Quotations in the table are drawn from FEMA 273 (Applied Technology Council, 1997).

Once performance objectives are quantified, they can be checked by engineering calculation. Holmes (2000) proposes six general requirements for a procedure to verify that a design (new or retrofit) meets PBD objectives. It must (1) accommodate any ground motion as input, (2) consider structural degradation and duration of ground motion, (3) model ductile and brittle elements, (4) model casualties, repair costs, and downtime, (5) the reliability of its outputs must be explicitly stated, and (6) it must have industry consensus. Because ABV produces detailed assembly damage statistics, it can meet many of these requirements, and at present only lacks a methodology to estimate casualty risk, and industry consensus.

Table 1. Illustrative translations of qualitative performance terminology. The qualitative term used in a PBD code is shown in the left column. A reasonable numerical translation and an example are shown in the second and third columns.

Qualitative term	Translation	Example
Negligible, few, little	0 - 1%	"Generally negligible [ceiling] damage:" less than 1% of ceiling area is damaged.
Some, minor	1 – 10%	"Some cracked [glazing] panes; none broken:" Between 1% and 10% of lites visibly cracked; no glass fallout.
Distributed	10 – 30%	"Distributed [partition] damage:" between 10% and 30% of partitions need patching, painting or repair, measured by lineal feet.
Many	30 – 60%	"Many fractures at [steel moment frame] connections:" between 30% and 60% of connections suffer rejectable damage.
Most	60 – 100%	"Most [HVAC equipment] units do not operate:" at least 60% of HVAC components inoperative.

FRAMEWORK FOR GATHERING EARTHQUAKE DAMAGE STATISTICS

The ABV approach also suggests a framework for systematically gathering damage data during earthquake field surveys. Much of the literature on earthquake experience focuses on identifying what can happen, that is, it identifies damage modes and explains causes of component failure. Quantitative statistics in field surveys are often limited to macroscopic effects: number of housing units lost or bridges damaged.

While this information is valuable, it would also be valuable to gather fragility data, that is, to quantify the relationship between the seismic demand and the probability that components will fail when subjected to that level of demand. A useful procedure to gather fragility data must include four crucial features:

1. Standard component names. Components should be categorized using a generally accepted taxonomic system, so that data from different locations and earthquakes are readily comparable with each other.
2. Standard failure modes. Failure modes should be described relative to important and widely understood performance goals. For example, failure could be described using the performance goals described in FEMA 273 (Applied Technology Council, 1997).
3. Standard quantification of the structural response. Each component must be associated with a level of seismic demand to which it was subjected. The demand to which components were subjected is often evident from nearby damage or can be estimated with later structural analysis.
4. Damage ratio. For each assembly type and damage mode, one must know both how many failed and how many did not fail, at a given level of seismic demand. Often the total number (failed plus not failed) is missing from survey data.

The ABV framework includes all four features, and can provide a pattern for gathering earthquake data that can be readily understood by researchers and directly used in loss estimation and risk management. ABV's taxonomic system is extended from a widely used assembly category system. It uses failure modes that are directly relevant to performance goals and repair costs. Each assembly is associated with well-understood structural response parameters that can be directly calculated from a structural analysis. Finally, assemblies are quantified in well-defined units that make calculation of damage ratio straightforward.

ILLUSTRATION OF ABV WITH AN EXAMPLE BUILDING

EXAMPLE BUILDING DESCRIPTION

The ABV method is generally applicable to any building whose assembly fragilities can be characterized as a function of structural or ground motion parameters. The approach is illustrated here using the example of a hypothetical Los Angeles office building. As shown in Figure 3, the building is a three-story pre-Northridge welded steel moment frame (WSMF) structure, with the WSMFs located at the perimeter. The perimeter frame shown in the elevation was designed for the SAC project to meet pre-Northridge standards in Los Angeles. The plan and nonstructural design aspects were developed for the present study.

The building has three 30-ft. bays in each direction plus 10-ft chamfers at each end. Beams are A36 steel. Columns are A572 Grade 50. Diaphragms are concrete topping on metal deck. Interior partitions are constructed of gypsum board over 3-5/8 in. metal studs with wallboard screws. The exterior is clad with lightweight glazed aluminum panels with gypsum board on the interior side. Ceilings are constructed of a suspended aluminum T-bar system with lay-in tiles and fluorescent lighting fixtures and perimeter attachment. Two gearless traction elevators provide vertical transport. Firm soil is assumed. Mean values are used for yield strength instead of nominal values. No splice or doubler plates are used.

Columns are fixed at the base. Dimensions are centerline. Beams are modeled as elastic elements with nonlinear springs at each end. The contribution of interior gravity frames to structural the response is accounted for by an additional column (column line E in Figure 3) tied to the frame by a rigid link. The building houses commercial office space. Monthly rental income is $2.50/sf net, i.e., calculated based on tenant square footage exclusive of common spaces. The ground floor is shared by office space, a lobby, and building service equipment. Upper stories are wholly devoted to office space. The total building replacement cost is estimated to be $4.9 million.

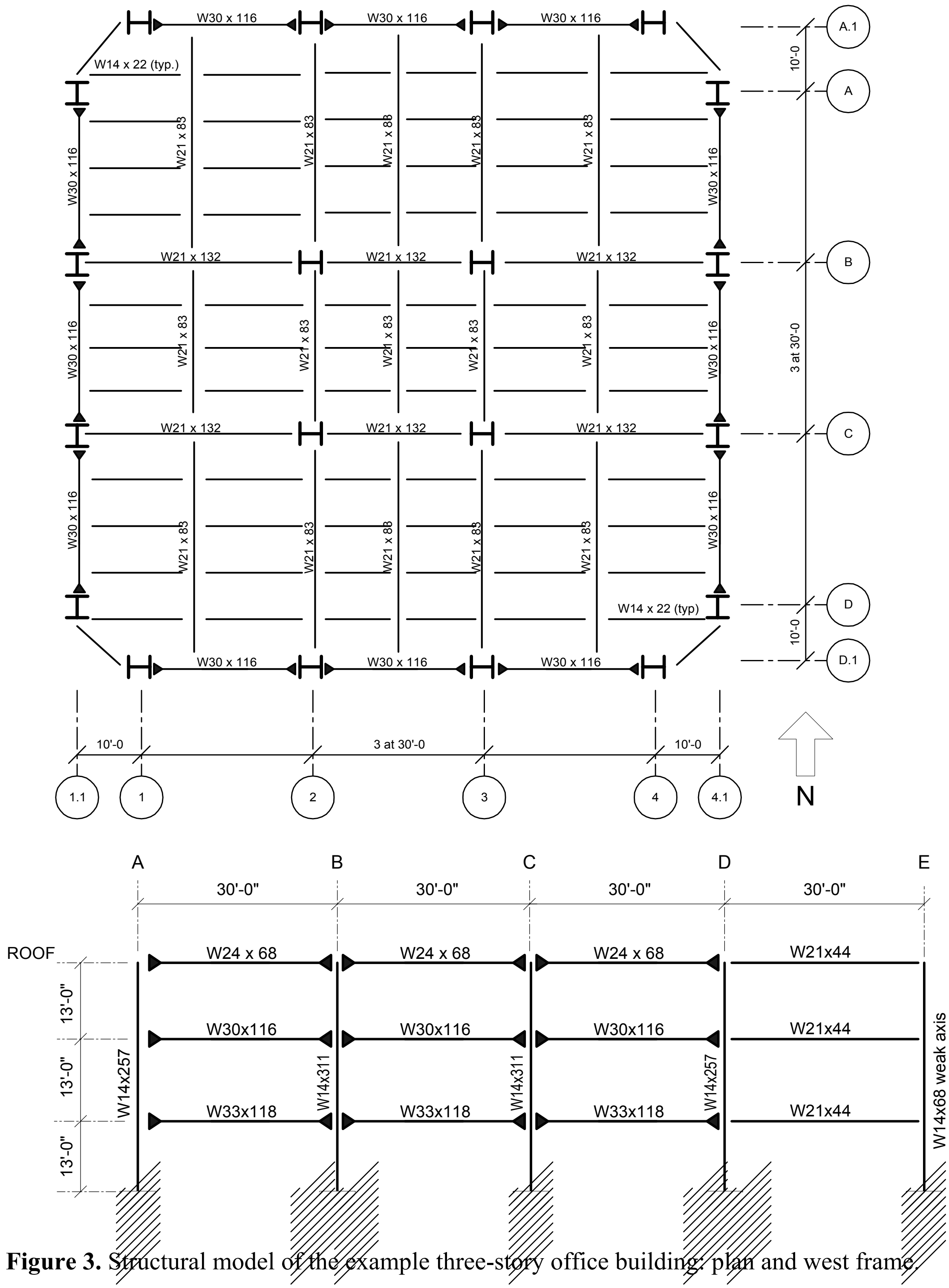

Figure 3. Structural model of the example three-story office building: plan and west frame.

CHARACTERIZATION OF GROUND MOTION

A set of earthquake ground motions were created by Somerville et al. (1997) for the project on steel structures supported by the Federal Emergency Management Agency (FEMA), referred to as the SAC-steel project. Some of the ground motions are scaled versions of real earthquakes; others were created using broadband simulation methods. All of the ground motions were scaled to approximate spectral ordinates from the 1996 United States Geological Survey probabilistic ground motion maps in the period range of 0.3 to 4 seconds (Frankel et al., 1996). For this study, we used 40 of the ground-motion records created for the Los Angeles area. For computational convenience, only ground motions with a 0.02-second sampling rate were used.

ASSEMBLY FRAGILITY AND COST FUNCTIONS

Table 2 and Table 3 list the assemblies considered in the sample application, their capacity data and repair-cost data. Table 2 lists the damageable assemblies used in the building, how they are quantified in the inventory, their damage states, the relevant response parameter, and the median and logarithmic standard deviation of the capacity to resist that damage state. The median capacity is denoted by x_m, the logarithmic standard deviation by β. A lognormal distribution is used for each capacity X_d, as shown in Equation 13, in which X_d represents the capacity to resist damage state d, and $\Phi(s)$ represents the cumulative standard normal distribution evaluated at s. The unit repair costs and repair durations shown in Table 3 are quantified the same way, and the random variables are likewise assumed to be lognormally distributed with the parameters shown.

$$F_{Xd}(x) = \Phi\left(\frac{\ln(x/x_m)}{\beta}\right) \tag{13}$$

The WSMF connection capacity is derived from data published in SAC (1995a). Rihal's (1982) data were analyzed to develop capacities for metal-stud drywall partitions. Laboratory tests performed by Behr et al. (1998) were used to describe glazing capacity. Capacities for electrical components come from Swan at al. (1998). Sprinkler capacities were derived from Porter et al. (1998). Suspended ceiling capacity was derived using reliability methods, based on the geometry and material properties of the components that constitute a suspended ceiling. The interested reader is referred to Porter (2000) for the derivation of these capacity distributions. No judgment-based fragility functions were used in the analysis of the example building.

Median repair costs and durations for architectural elements and for sprinklers were calculated based on RS Means (1997b). Repair costs and durations for WSMF connections based on replacing damaged moment connection with SAC (1995b) haunched WT fixtures. The distribution of repair cost for this fixture is calculated from an unpublished construction cost estimate created for the owner of a large steel-frame building damaged in the Northridge earthquake. All other cost and time parameters were estimated by judgment. While published data on these items were not available to the present author, they should be readily available to engineering firms familiar with post-earthquake repairs. For the present illustration, they were estimated conservatively and are shown in italics to indicate lower confidence in their accuracy.

The numbers in Tables 2 and 3 should be viewed in the light of their intended purpose: to illustrate the general ABV framework, rather than to present definitive fragility and cost distributions. The research emphasis at this stage is development of the methodology, rather than an attempt to populate a large library of fragility and cost functions.

Table 2. Assembly capacity. The table shows median and logarithmic standard deviation of capacity for the damageable assemblies in the example building, (x_m and β, respectively) in terms of the relevant structural response: peak ratio of beam-end elastic moment to yield capacity (demand-capacity ratio, or DCR), peak transient drift ratio (TD), and peak diaphragm acceleration (PDA).

Assembly	**Unit**	**Damage state**	**Response parameter**	x_m	β
WSMF connections	Bm-col. Conn.	SAC damage (1)	DCR	1.6	1.7
WSMF connections	Bm-col. Conn.	Same, > W1, C1(2)	DCR	3.3	1.8
Glazing	30 sf pane	Cracking	TD	0.040	0.36
Glazing	30 sf pane	Fallout	TD	0.046	0.33
Drywall partition	8'x8'	Visible damage	TD	0.0039	0.17
Drywall partition	8'x8'	Signif. damage	TD	0.0085	0.23
Acoustical ceiling	One room	Collapse	PDA	$46/x_s$(3)	0.80
Unbraced sprinklers	12 lf pipe	Fracture	PDA	4.2*g*	0.87
Braced sprinklers	12 lf pipe	Fracture	PDA	8.4*g*	0.87
Low volt. switchgear	Set	Inoperative	PDA	1.1*g*	0.64
Med. volt. switchgear	Set	Inoperative	PDA	1.6*g*	0.80
Motor installation	Set	Inoperative	PDA	0.79*g*	0.52
Generator	Set	Inoperative	PDA	0.87*g*	0.51

(1) SAC (1995a), pg. 7-33, including G, C, W, S, and C, excluding P (panel-zone damage)
(2) i.e., the same SAC damage states, except that the incipient damage states W1 and C1 are not included
(3) x_s = (ceiling length + width)/2, ft. The result is in terms of gravity, *g*.

Table 3. Assembly repair cost and repair duration. The table shows median and logarithmic standard deviation (x_m and β, respectively) of repair cost per damaged assembly, and of total hours to repair a damaged assembly. Costs are for a building in the city of Los Angeles, in 1997 dollars.

			Cost/unit ($)		**Time/unit (hr)**	
Assembly	**Repair**	**Unit**	x_m	β	x_m	β
WSMF connections	WT(1)	Conn.	23,900	0.58	8	0.58
Glazing	Replace	30-sf pane	439	0.26	0.4	*0.5*
Drywall partition	Patch	8'x8'	50	*0.5*	0.4	*0.5*
Drywall partition	Replace	8'x8'	192	*0.5*	1.5	*0.5*
Acoustical ceiling	Replace	sf	2.21	*0.5*	0.016	*0.5*
Unbraced sprinklers	Replace	12 lf	156	*0.5*	1	*0.5*
Braced sprinklers	Replace	12 lf	156	*0.5*	1	*0.5*
Low volt. switchgear	Anchor(2)	Set	*1000*	*1.0*	*8*	*1*
Med volt. switchgear	Anchor(2)	Set	*1000*	*1.0*	*8*	*1*
Motor installation	Anchor(2)	Set	*1000*	*1.0*	*8*	*1*
Generator	Anchor(2)	Set	*5000*	*1.0*	*16*	*1*

(1) Replace moment connection with SAC (1995b) haunched WT
(2) Service existing equipment, restore to its original position, and install seismic anchorage

EXAMPLE BUILDING VULNERABILITY FUNCTION

For each S_a value in {0.1, 0.2, … 1.5g}, twenty simulations of ground motion, structural response, damage state, repair cost, and loss-of-use duration and cost were performed. Since a nondegrading structural model was used, the analysis was limited to values of $S_a \leq 1.5$g, an approximate upper bound of validity. At higher values, local and global collapses are increasingly likely and would not be reflected by the structural analysis. The emphasis of the present analysis is therefore on lower levels of damage. Figure 4 shows the resulting loss amounts as a fraction of building replacement cost. The solid line fit to the data represents mean vulnerability. The mean residual coefficient of variation, denoted by $\delta_{y|x}$, was calculated for the example building to be 0.60.

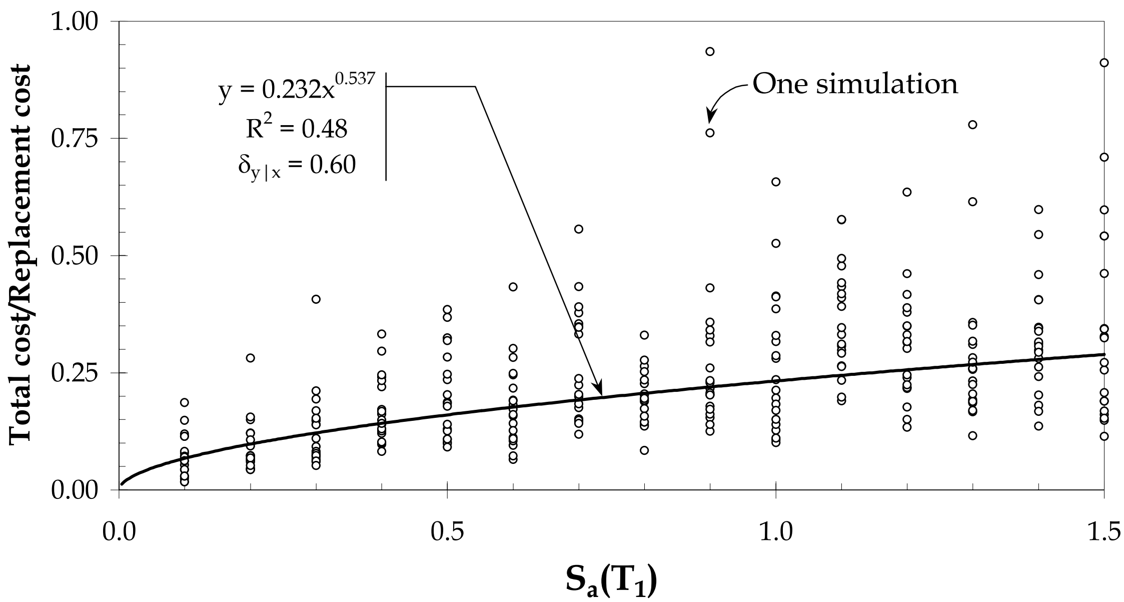

Figure 4. Seismic vulnerability function for the example building. Each dot represents one simulation of ground shaking, structural response, damage, and repair. $S_a(T_1)$ measures spectral acceleration at the building's fundamental period. Total cost refers to repair plus loss of use. Twenty simulations are shown per increment of S_a. The solid line represents a best fit on the mean.

The results shown in Figure 4 represent slow-repair assumptions: change-of-trade delays are taken as R_T = 14 days, and only one crew is available for each repair task. That is, a single crew repairs a particular assembly type in operational unit m = 1, then immediately moves to operational unit m = 2, etc. Two weeks pass before the next trade arrives to commence work on operational unit m = 1.

Losses for a fast-repair assumption have also been calculated, where R_T is assumed to be 2 days, and all operational areas are assumed to undergo repairs in parallel. That is, enough crews are available so that all operational units may undergo repairs on the same assembly type simultaneously. This assumption leads to a vulnerability function that is somewhat lower than the slow-repair function because of smaller loss-of-use costs. Thus, the total vulnerability for the sample building appears to be modestly sensitive to the speed at which repairs are performed.

Figure 4 shows that there is substantial uncertainty in the damage factor: on average, the residual coefficient of variation on loss is 0.60. This is the accumulated uncertainty resulting from four explicitly considered sources of variability:

- Uncertain ground motion time history for a given spectral acceleration;
- Uncertain assembly damage given the structural response to the ground motion;
- Uncertain repair costs for given damages; and
- Uncertain productivity of repair crews.

The results shown in Figure 4 omit uncertainty on structural response given the ground motion, which could theoretically be included in the analysis, but in this application was not. Uncertain structural response would increase total uncertainty by some unknown amount, perhaps to 0.70 or 0.75.

This uncertainty does not mean that the ABV approach is no good, that it produces too much uncertainty to be useful in loss estimation and risk management. Rather, it indicates that even with highly detailed information on ground motion, assembly damageability, and so on, there remains substantial uncertainty in total loss. Furthermore, it represents a lower of uncertainty for category-based methods, which use less detailed information in order to model a wider variety of similar buildings.

EXAMPLE BUILDING PERFORMANCE EVALUATION

The example building was analyzed for the component damage objectives shown in Table 4. In the table, "damage ratio" is defined as the fraction of such components damaged; the figures shown in the last two columns represent the maximum allowable damage ratio, as interpreted from FEMA 273 (Applied Technology Council, 1997) according to the translations shown in Table 1.

The life-safety (LS) performance level was tested for a ground shaking level of S_a = 1.5g, equivalent to the level of shaking associated with 10% probability of exceedance in the next 50 years for the selected Los Angeles site. Immediate occupancy (IO) was tested using S_a = 0.8g, which is the level of shaking with 50% exceedance probability in the next 50 years.

Figure 5 shows estimated probabilities of achieving the performance objectives for this building, based on 100 simulations for each earthquake level. That is, the figure shows the fraction of simulations in which the simulated damage ratio was less than the allowable damage ratio of Table 4.

The figure shows that the building almost certainly fails both overall life-safety (LS) and immediate occupancy (IO) requirements, primarily because of the fragility of the WSMF connections and the acoustical ceiling. The fragility of unanchored switchgear also prevents the building from passing immediate occupancy performance requirements.

Table 4. Sample maximum allowable life-safe (LS) and immediate-occupancy (IO) performance levels. The table shows the maximum fraction of assemblies reaching or exceeding the named damage state that would qualify as passing the stated performance levels.

Assembly	Damage state	Unit	Max damage ratio LS	Max damage ratio IO
WSMF connections	Fracture	ea	0.01	0.00
Exterior glazing	Visible cracking	lite	0.30	0.10
Drywall partition	Noticeable damage	8 lf	1.00	0.10
Acoustical ceiling	Collapse	ea	0.30	0.01
Traction gearless elevator	Inoperative	ea	1.00	0.00
Gas water heater	Inoperative	ea	1.00	0.00
Wet-pipe sprinkler system	Leakage	12 lf	0.10	0.01
Building heating terminal unit	Inoperative	ea	1.00	0.00
Chilled water cooling tower	Inoperative	ea	1.00	0.00
Switchgear – low voltage	Inoperative	ea	1.00	0.00
Switchgear -- medium voltage	Inoperative	ea	1.00	0.00
Motor installation	Inoperative	ea	1.00	1.00
Generator	Inoperative	ea	1.00	1.00

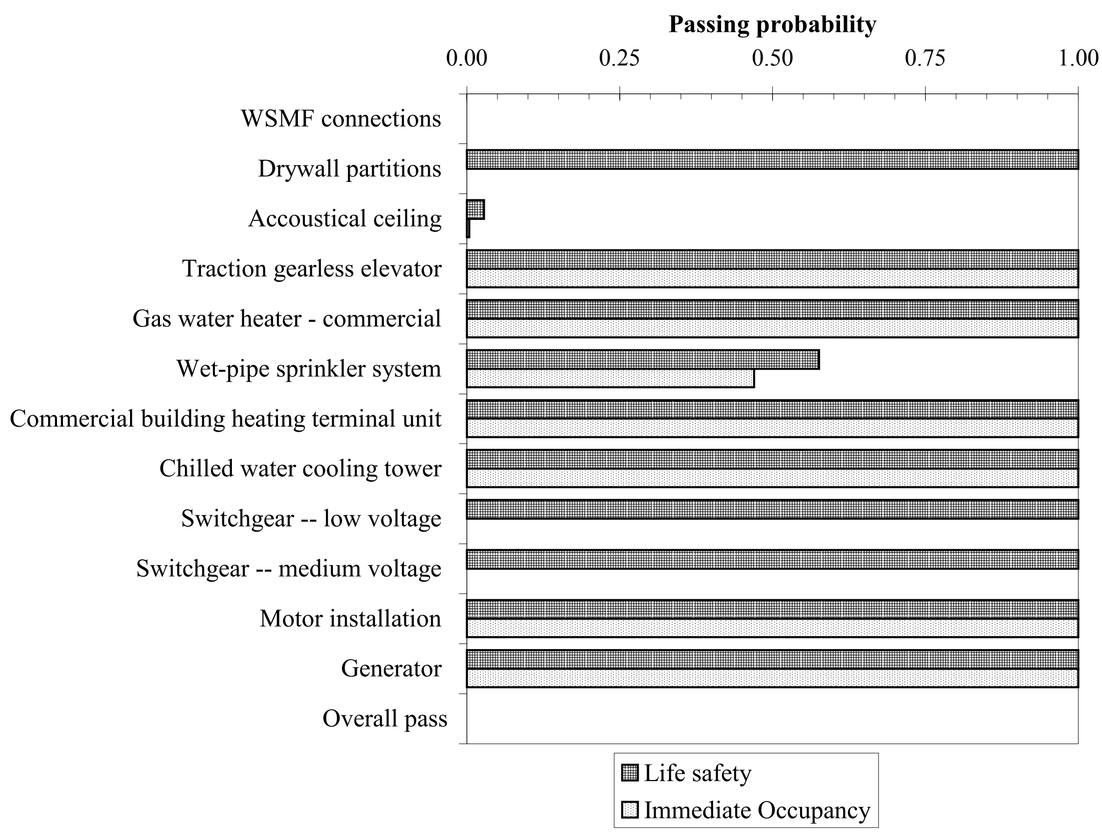

Figure 5. Probability of the example office building meeting sample performance levels. Each pair of bars shows the probability that a particular assembly will pass its performance objectives under the life-safety (LS) and immediate occupancy (IO) performance levels.

CONCLUSIONS

This paper has summarized a rigorous methodology for developing building-specific seismic vulnerability functions. The development of the methodology is detailed in Porter (2000). The approach, entitled assembly-based vulnerability (ABV), accounts for the detailed structural and nonstructural design of a building, rather than relying on category-based vulnerability functions that are more appropriate for use in macroscopic (multi-building) loss estimation.

The vulnerability functions developed with ABV are probabilistic and can account for uncertainty in ground motion, structural response, assembly fragility, repair cost, repair duration, and loss due to downtime. Because the methodology produces detailed damage simulations at the assembly level, the analyst can calculate the probability that a particular building will meet detailed performance-based design objectives. The assembly-level resolution also means that a designer or analyst can examine the benefits of alternative design, rehabilitation, or retrofit details, to which a category-based approach is not sensitive.

Each module of the approach can be implemented though state-of-the-art engineering methods and data. This modularity means that an analysis based on ABV can be improved incrementally as new data or structural analysis tools become available. For example, one can use new fragility information on one particular assembly type to improve the accuracy of an entire building vulnerability function, without changing other aspects of the model.

These benefits come at a cost: an ABV analysis involves all the work of a structural analysis, plus the creation of an inventory of building assemblies and the compilation of assembly capacity and repair-cost distributions. While the capacity and cost distributions can be reused in later analyses, they must first be created from laboratory or earthquake experience data or from theoretical fragility models, and compiled in a central library. The computational aspects of the approach however can be readily automated.

Because of its reliance on structural analysis of single buildings, the method is not intended to model the seismic vulnerability of entire classes of buildings in the manner of ATC 13 (Applied Technology Council, 1985) or HAZUS (National Institute of Building Sciences, 1997). However, if applied to a wide variety of particular buildings selected to represent the diversity of existing construction, ABV could be used to produce vulnerability functions for common generic structural types.

It is not known whether ABV produces loss estimates that are more accurate for a particular building than a category-based approach would produce. If "more accurate" means a smaller uncertainty on loss, then the ABV approach probably meets that criterion, since the uncertainty it produces must be a lower bound for a category-based vulnerability function that refers to a broad category of similar buildings, expert opinion notwithstanding on the uncertainty of the category-based vulnerability function. (As noted above, expert opinion typically underestimates uncertainty unless it is elicited very carefully.)

If "more accurate" means that the mean squared error from ABV is less than the mean squared error from a category-based approach, then the question can be answered, but at great cost. Answering it would require performing a large number of ABV analyses on a variety of buildings, and then waiting for an earthquake, or by performing shake-table tests of a large number of full-scale buildings.

In the end, an ABV model is only as accurate for a particular building as its constituent ground motion recordings, structural model, and assembly capacity and cost distributions. These at least are within the control of the analyzing engineer, who also has access to the building and its details. This is in contrast with an answer provided by experts years ago who were thinking of other buildings under other conditions with other objectives in mind.

GLOSSARY

ABV: Assembly-based vulnerability, a framework for developing building-specific seismic vulnerability functions.

ARMA: autoregressive moving average, a method to generate a random, artificial time series such as an earthquake accelerogram. ARMA models can include time-varying amplitude and variance similar to an original time series.

ATC 13: A document by the Applied Technology Council (1985) that, among other contributions, presents seismic vulnerability functions for a wide variety of structure types.

Conditional cumulative probability distribution: denoted generically by $F_{X|Y}(x|y)$, it gives the probability that X will take on a value less than or equal to x, given that the uncertain variable Y takes on the particular value y.

Conditional probability mass function: denoted generically by $p_{X|Y}(x|y)$, it gives the probability that the uncertain variable X will take on the particular value x, given that the uncertain variable Y takes on the particular value y.

Cumulative probability distribution: denoted generically by $F_X(x)$, it gives the probability that an uncertain variable X will take on a value less or equal to a particular value x.

HAZUS: A standard methodology and its associated software implementation for estimating economic and other loss associated with earthquake shaking. (Other perils such as wind and flood are also addressed by HAZUS.) See National Institute of Building Sciences (1997).

Gantt chart: a chart that depicts tasks to be performed as horizontal bars whose length indicates the duration of each task. Tasks are connected by lines that indicate the order in which the tasks must be accomplished.

PBD: performance-based design, a design philosophy that seeks to ensure that a building will meet certain observable performance goals after being subjected to given levels of ground shaking.

Seismic vulnerability function: a motion-damage relationship between financial or other loss and some measure of shaking intensity.

WSMF: welded-steel moment frame.

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Appendix E. Assembly Fragilities

Under the ABV framework, each damageable assembly must be associated with one or more fragility functions, that is, relationships that give the probability that the assembly will be damaged in some predefined way when subjected to a certain amount of structural response. The development of assembly fragilities is summarized here. The non-technical reader can skip this appendix; it is presented for completeness, and to illustrate the nature of the testing and analyses needed to produce fragility functions.

Cement Stucco Walls

Chai *et al.* (2001) performed in-place shear tests of 28 12-ft. specimens of cripple walls of varying configurations. The cripple walls had a stucco finish on one side (two 3/8" coats) and OSB (oriented strandboard, a material similar to plywood) sheathing on the other. The authors found that, after some initial cracking occurred, no widespread damage to the stucco finished occurred despite drift ratios that were large enough to cause the structural failure of the wall, namely, nails tearing through OSB edges and failure of stud connections. The connection between the wire mesh and the stud wall appeared to act as a fuse, breaking the link between the stucco and the wall at a point after minor, repairable cracking of the stucco occurred but before extensive cracking could occur.

Pardoen *et al.* (in progress) present results of a series of 108 in-plane racking tests of 8-ft-by-8-ft light-framed shearwalls of varying sheathing, nailing, and material configurations. The primary objective of these tests was to determine load-displacement characteristics for the various configurations, rather than to relate observable damage to load or displacement. A variety of strength and stiffness data are provided. Most relevant for present purposes are six tests, numbers 20A, B, and C, and 21A, B, and C, in which walls were tested that had stucco on one side, and no other sheathing. These are similar to the unsheathed stucco walls in the index buildings for the present project. The authors make no observations of physical damage, but they do provide the load and displacement corresponding to yield and maximum strength are reached for these six tests, i.e., the yield limit state and strength limit state. If these limit states, which are defined in terms of the load-displacement relationship, can reasonably be associated with physical damage, the data can be used to inform the fragility functions used in the present study. An analysis of these data led to the conclusion that it is reasonable to consider three limit states.

Note that Osteraas (2002) has performed laboratory tests of woodframe walls with wallboard interior finish and stucco exterior finish, but the results of these tests are unavailable at the time of this writing.

Cracking of stucco. This limit state is associated with minor cracking of the stucco. A reasonable fix is to patch any cracks and repaint the wall. Based on the Chai *et al.* (2001)

data, this limit state occurs at peak transient drift with median x_m = 1.2% and logarithmic standard deviation of 0.48. This limit state can occur in stucco walls with properly embedded lath, but apparently not otherwise.

Fracture of connection between stucco and studs. This limit state can occur before or after limit state 1 is exceeded, depending on how well the lath is embedded in the stucco. In all six tests performed by Pardoen *et al.* (in progress), limit state 2 occurred without any cracking of the stucco. The tests indicate a median capacity of approximately x_m = 1.2% drift. The logarithmic standard deviation will be taken as 0.50. Repair requires removal and replacement of the stucco. This limit state can occur in stucco walls with improperly embedded lath, but apparently not otherwise.

Collapse of cripple wall. Although minimal data are available, the tests by Chai *et al.* (2001) indicate that a 2-ft. cripple wall subjected to dynamic loading collapses at 2 in. of drift, or 8.3%. Stucco cripple walls will therefore be taken as having a median capacity to resist collapse of 8.3% drift, (2 in drift over a 24-in. height) with a logarithmic standard deviation of 0.10. Collapse can occur regardless of embedment of the lath. Inadequate data are available to generalize these results to taller stucco walls.

Drywall Partitions And Drywall Ceiling

McMullin *et al.* (2001) review previous experimental data and present the results of 17 new laboratory tests of gypsum wallboard partitions. Specimens were 8 ft. high, 16 ft long, sheathed on both sides with ½-in., standard-grade gypsum wallboard. Wallboard was oriented horizontally. The specimens were all framed with 2x4 nominal dimension lumber of grade 2 or better Hem-fir. Specimens have double top plates, single bottom plates, 4x4 endposts, and intermediate 2x4 studs at 16 in. OC. A variety of configurations of window and door openings were used. The authors performed in-plane racking tests of cyclic, pseudostatic loading, without gravity loading on the top plate. Various combinations of fastener types and spacing were examined. The study documents load-displacement behavior, and notes damage and required repairs as functions of peak drift.

Three construction contractors were employed to determine the repairs that would be required if they had observed the damaged walls in actual practice, i.e., in a damaged house. Several failure modes were observed, including (among others): cracking of wallboard at corners of window and door openings; cracking of compound or paint at fastener heads; cracking of taped joints; local buckling of panels at opening corners; crushing of wallboard at the wall perimeter; and bulging of large regions of wallboard panels.

Despite the variety of failure modes, the authors find that three limit states exist: minor damage, associated with minor cracking; moderate damage, which requires extensive repair, and severe damage, which requires demolition and replacement of the wall. The transition from one limit state to the next appears to follow a step function, i.e., there is no gradual transition from one damage state to the next. The reason is that the limit states are in practice defined in terms of the size and nature of the repair crew. The minor damage state can be repaired by a handyman. The partition enters the moderate damage state when repair requires carpenters and a paint crew. The third damage state requires all the trades involved in replacing a wall, including electricians and plumbers as appropriate.

Note that the loss estimation subcontractor for the present project (Boyd) disagreed that the minor damage state would not require a paint crew; he estimates the cost to repair the light damage state as equal to that of the moderate damage state, and believes that in both cases touch-up paint could not be blended adequately to the undamaged wall, and that a paint crew would be required to paint the entire room. For cost-estimation purposes, then, the two limit states are identical.

Detail on the fragility functions for these limit states is now presented.

LS1, paint damage at fasteners heads and wallboard cracks at openings. Deformation of fasteners causes paint to crack over the fastener head. In the McMullin *et al.* (2001) tests, paint damage occurs in dynamic tests at a median drift ratio of 0.25%, with a coefficient of variation of approximately 0.53, as shown in Figure E-1. (The figure shows the x-axis measuring drift in radians, but this is clearly a typographic error. The horizontal axis measures drift either in inches, drift ratio in percent, or hundredths of a radian, all of which would be nearly equal for an 8-ft. wall.) At approximately the same level of drift, cracks appear in GWB at the corners of door and window openings.

McMullin *et al.* (2001) observe that in tests of walls sheathed only with GWB (no structural sheathing), “Almost every test has reached this event by a drift of approximately 0.5%.” They provide a fragility curve, copied to Figure E-2, that suggests cracking occurs between 0.1 and 0.5 radians, or 10% to 55% drift, but again this is a typographic error, and the data are meant to show a median capacity of approximately 0.22% drift and a coefficient of variation of approximately 0.38. Repair for both types of damage requires patching and repainting. Because they occur at approximately the same level of drift, and require approximately the same repair effort, these two damage modes can for convenience be modeled using a single fragility function, with median capacity x_m = 0.25%, and logarithmic standard deviation β = 0.5.

LS2 Wallboard fastener failure (tearthrough), and cracking of wallboard joints. McMullin *et al.* (2001) identify a limit state at which cracking occurred at vertical joints to an intersecting wall, suggestive of fracture of the fasteners, fastener pullout, or fastener tear-through. Repair requires tape, mud (i.e., application of joint compound), sand and paint. (See Figure E-3. “Radians” on the *x*-axis is again a typo. Read “percent drift”.) This limit state occurs at a mean value of 0.45% drift and a standard deviation of 0.22% drift, indicating a median capacity of 0.40% and logarithmic standard deviation β = 0.5.

LS3 Collapse. Based on Pardoen *et al.* (in progress) test groups 7 and 8, the collapse limit state (CLS) is taken as having median capacity of x_m = 1.6% drift and logarithmic standard deviation $\beta = 0.2$. Collapse is assumed to require demolition and replacement of the wallboard.

Figure E-1:
Fragility of Paint Cracking over Wallboard Fasteners (McMullin *et al.*, 2001).

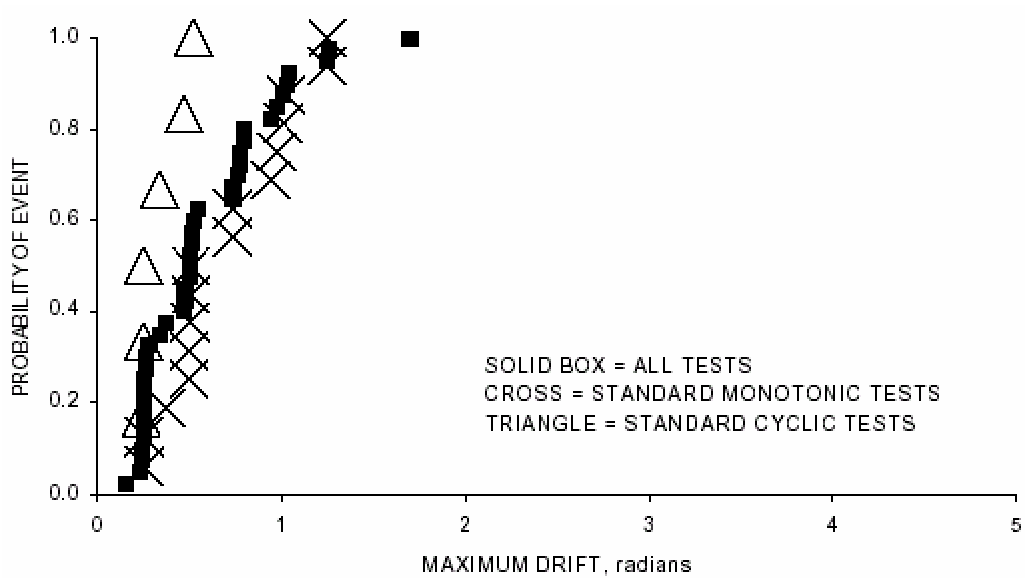


Figure E-2:
Fragility of wallboard cracking at wall openings (McMullin *et al.*, 2001).

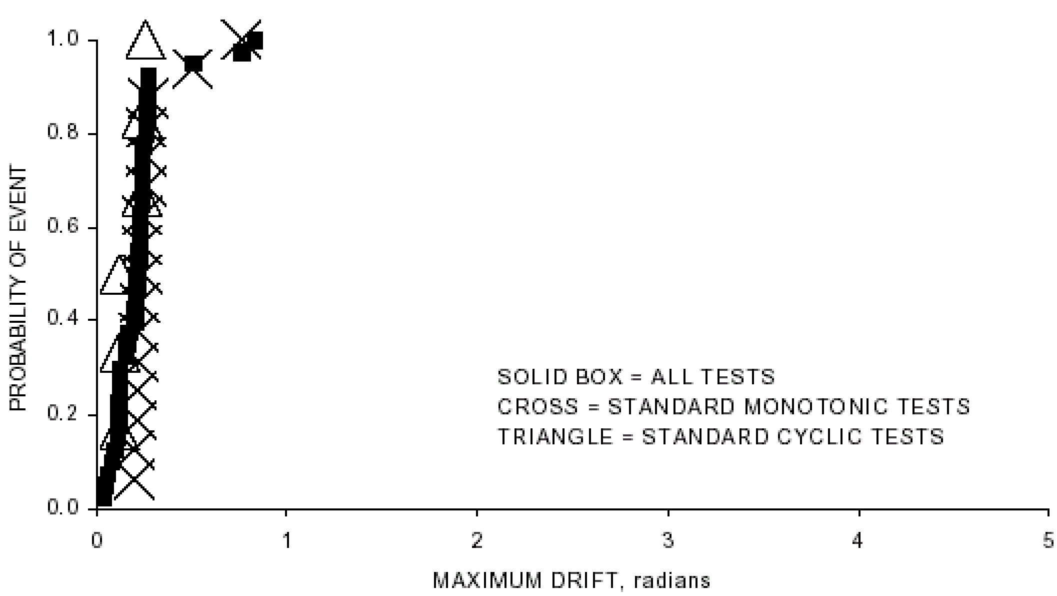
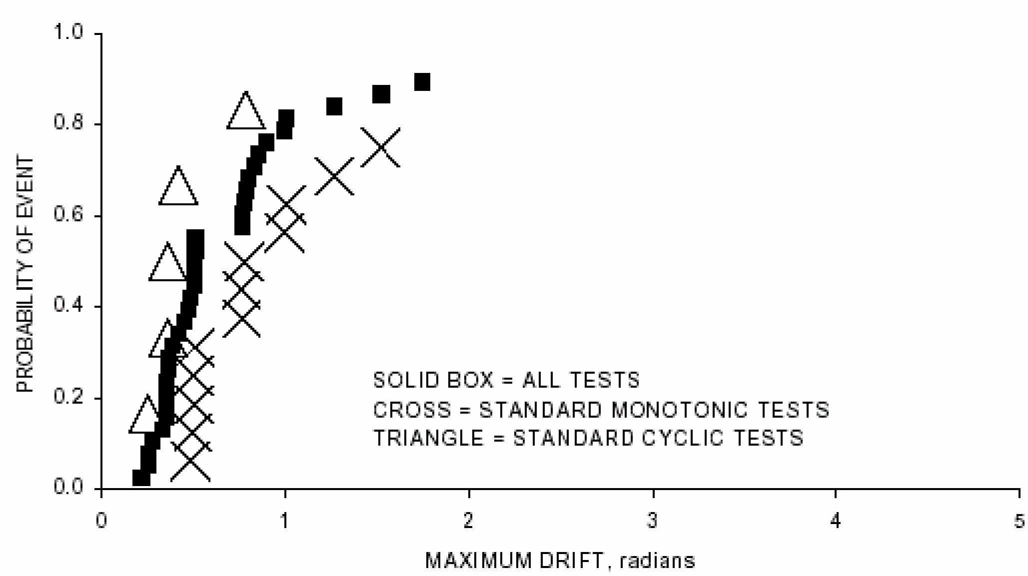


Figure E-3:
Fragility of Wallboard Cracking at Vertical Joint to Intersection Walls
(McMullin *et al.*, 2001).



Plywood and Oriented Strandboard (OSB) Sheathing Without Stucco

Pardoen *et al.* (in progress) present results of woodframe shearwall racking tests for variety of wall specimens. They report on 27 groups of tests, each group comprising destructive tests of three specimens. Each specimen comprises one 8-ft.x8-ft. panel. In the case of plywood, OSB, and wallboard sheathing, two vertical 4-ft.x8-ft. sheets are used. Table E-1 summarizes relevant test groups.

The testing protocol consists of approximately 40 to 100 cycles of displacement-controlled racking in both directions, with results shown in terms of load-displacement behavior. No detail is available yet regarding physical damage. The authors present load and displacement for two limit states: yield and strength. The yield limit state (YLS) is probably associated with the onset of significant stiffness degradation of the connections, but short of pullout or tearthrough, such degradation does not indicate a level of damage that requires repair. The YLS therefore is not of interest to the present study. The strength limit state (SLS) is the point on the backbone load-displacement curve associated with maximum force. One additional point on the load-displacement curve is of interest.

Table E-1:
Shearwall Assembly Varieties (Pardoen *et al.*, in progress) and Fragility Parameters.

Test Group	Sheathing Thickness, Mat'l, Application			Nails (hand-driven common)	Nail & Nailing Info (in)						Limit state drift ratios, mean and C.O.V. (1)			
					Size		Spacing		Edge Dist		m_{SLS}	d_{SLS}	m_{CLS}	d_{CLS}
1	3/8	STR I	one side	8d	2 1/2	.135	6	12	1/2	3/4	0.014	0.10	0.022	0.01
2	3/8	STR I	one side	8d	2 1/2	.135	4	12	1/2	3/4	0.019	0.14	0.024	0.01
3	15/32	STR I	one side	10d	3	.152	6	12	1/2	3/4	0.018	0.18	0.030	0.01
4	15/32	STR I	one side	10d	3	.152	4	12	1/2	3/4	0.020	0.11	0.028	0.03
5	3/8	STR I	one side	8d	2 1/2	.135	4	12	3/8	3/8	0.013	0.06	0.020	0.25
9	15/32	STR I	one side	10d	3	.152	2	12	1/2	1/2	0.020	0.06	0.031	0.22
10	7/16	OSB	one side	8d	2 1/2	.135	4	12	1/2	1/2	0.010	0.17	0.022	0.14
11	15/32	OSB	one side	10d	3	.152	6	12	1/2	1/2	0.010	0.12	0.020	0.05
12	15/32	OSB	one side	10d	3	.152	4	12	1/2	1/2	0.014	0.09	0.025	0.03
13	15/32	OSB	one side	10d	3	.152	2	12	1/2	1/2	0.014	0.09	0.025	0.52
25	3/8	STR I	one side	8d	2 1/2	.135	2	12	1/2	3/4	0.016	0.10	0.034	0.02
26	15/32	STR I	one side	8d	2 1/2	.135	4	12	3/8	3/8	0.013	0.06	0.016	0.05

1 SLS = strength limit state; CLS = collapse limit state; m = mean value of drift ratio; d = coefficient of variation of drift ratio

Though not addressed by Pardoen *et al.* (in progress), a collapse limit state (CLS) is often considered to occur when drift increases beyond the SLS, and the racking force decreases to 80% of the maximum value. The CLS data can be assessed from the Pardoen *et al.* (in progress) backbone curves. Results are shown in Table E-1. These data show that the drift at CLS is on average 1.9 times that of the SLS. The correlation is moderate, ρ = 0.65. This parameter would be relevant in a stochastic structural analysis to account for correlation between points on the load-displacement curves for shearwalls, and to account for correlated fragility parameters in the damage assessment. The present study does not take advantage of this knowledge of ρ, as will be discussed; it is provided for information only.

Note that it is possible to define other limit states than the YLS, SLS, and CLS. Pardoen *et al.* (in progress) focus on the YLS and SLS as particularly useful in defining a load-displacement backbone curve with a minimum number of control points. The question addressed now is whether important symptoms of physical damage can be associated with these features of the load-displacement curve, for use in coupling damage with engineering response.

The loss of stiffness and strength in wood-sheathed shearwalls is primarily associated with nonlinear behavior of connectors, including deformation of the hole, deformation of the connector, and connector pullout, pullthrough, or fracture. Uang *et al.* (2001) report that in shearwall tests performed at the University of California, San Diego, “As has been observed in previous shearwall testing, the sheathing panels rotated and the fasteners deformed as the displacement demand was increased." Their test results show that over a wide range of drift, virtually all displacement is associated with connections:

1. Anchorage connections (40-50%),
2. Nail slip (30-40%), and
3. Sill slip (10-20%)

This is not to imply that all physical damage of woodframe shearwalls is associated with connectors, or that all types of connectors will exhibit these failure patterns. Damage to framing members (studs, sill plates, and top plates) is not uncommon.

Deformation of the sheathing and chords contributes negligibly to deformation. Note that the Folz *et al.* (2001) model of wood shearwall in-plane racking (used in the present study to model the behavior of the shearwall elements of the index buildings) assumes this behavior. It assumes elastically shearing sheathing, pinned-end framing members and nonlinear connections, to which any damage occurs.

Since much engineering data on shearwalls is limited to load-displacement behavior, it is desirable to associate a point on the load-displacement curve at which some damage occurs necessitating repair. Two damage-oriented limit states seem reasonable. The first limit state of interest (let limit state 1 be denoted by LS1) is the point at which visible failure of connectors begins, requiring repair of the wall. The second limit state (LS2) is associated with the need to demolish and replace the wall. (In an entire building, this damage state in a single wall would probably coincide with similar damage to other wall segments in the same portion of the building, owing to displacement compatibility and similar fragility.)

Let $\underline{F}_1 = (X_1,Y_1)$ denote the drift and load at which LS1 is reached, and $\underline{F}_2 = (X_2, Y_2)$ denote the drift and load at which LS2 is reached. The present objective then amounts to quantifying $\underline{F}_1$ and $\underline{F}_2$. It will also be useful to denote the strength limit state by SLS, and to refer to the drift and load at which it occurs as $\underline{F}_{SLS} = (X_{SLS}, Y_{SLS})$, and to denote the collapse limit state by CLS, and to refer to it by $\underline{F}_{CLS} = (X_{CLS}, Y_{CLS})$.

A logical hypothesis is that LS1 occurs at the point where the shearwall achieves its maximum strength, i.e., the strength limit state (SLS) of the Pardoen *et al.* (in progress) tests. Let the hypothesis also include the assertion that LS1 occurs as a result of rotation of the sheathing panel such that one long edge of the sheathing panel remains attached to the framing, while the connections along the other long edge and two short edges deform and begin to fracture. This hypothesis can be tested by predicting which nails would likely fail, and by predicting the drift at which one would predict initial nail failure, and comparing that prediction with the SLS of tested specimens.

To test the hypothesis, consider first the kinematics of shearwall deformation. One can think of two extremes of the mode of deformation, A and B, illustrated in Figure E-4.

(Other kinematic modes are possible; these two are constrained to have the upper left-hand corner of the sheet coincide with the framing.) In case A (left), during drift the sheathing remains attached to one long edge, while connectors deform and fail along one long edge and two short edges. In case B, the sheathing panel remains attached along one short edge, while connectors deform and fail along two long edges and one short edge. The case that requires less deformation energy for a given level of drift should dominate.

Let D denote that portion of drift associated with deformation of the sheathing-to-framing connectors. Let δ denote the deformation of an individual connector. Let h represent height of the sheathing panel, and b the width of the sheathing panel. Let nail spacing at the edges be denoted by s. Ignore interior nailing.

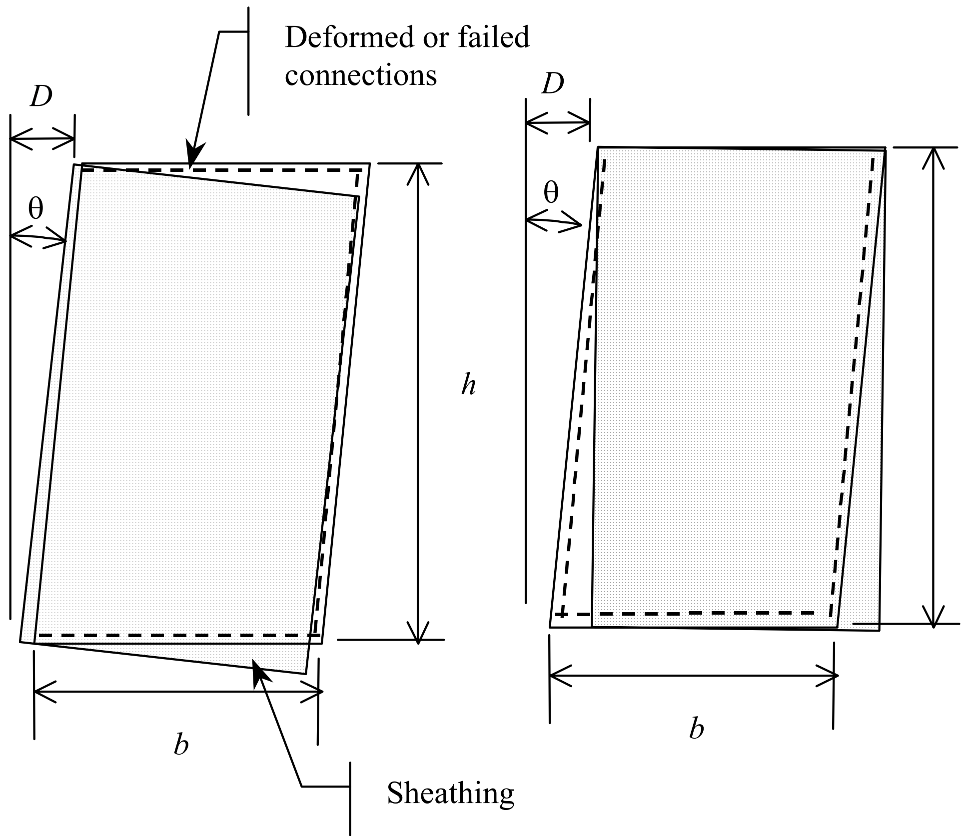
In case A, the maximum nail deformation equals Db/h, with the number of connections deforming being $(2b + h)/s$. A number of connections approximately equal to h/s along the long edge each deform by Db/h. Each of approximately $2b/s$ connections along the short edge deform an average of $Db/(2h)$. The total energy of deformation increases with increasing deformation of the connectors, so $\Sigma\delta$ can be used as an index of work. (Nonlinear force-deformation behavior of connectors makes this index only approximate). For case A, the index is calculated as shown in Equation E-1. In case B, the maximum nail deformation is D. On the long edges, $2h/s$ connectors deform an average of $D/2$, while on the short edge, b/s connectors deform D. The total work index of deformed connectors in case B is shown in Equation E-2. For $h>b$, $\Sigma\delta$ of case A is always less than that of case B for a given drift D. For $h = 2b$, the ratio of $\Sigma\delta$ for case A to that of case B is 0.5, as shown in Equation E-3, meaning case A requires less energy for drift D.

$$\Sigma\delta_A = h/s*Db/h + 2b/s*Db/(2h) = Db/s + Db^2/(sh) \tag{E-1}$$

$$\Sigma\delta_B = 2h/s*D/2 + b/s*D = D(h+b)/s \tag{E-2}$$

$$\Sigma\delta_A/\Sigma\delta_B = (\mathrm{Db/s} + \mathrm{Db}^2/(2\mathrm{sb}))/(3\mathrm{Db/s}) = 0.5 \tag{E-3}$$

Figure E-4:
Two Extreme Patterns of Sheathing-to-Framing Connection Failure.



This leads to the prediction that destructive tests should show connection failure predominantly along both short edges of the sheathing, and one long edge. In support of the hypothesis, virtually all test specimens (other than stucco-finished) examined by Uang *et al.* (2001) exhibit this failure pattern. (Stucco crosses the edges between sheathing panels, and acts as a restraint that changes the work required to impose drift, so the hypothesis would not apply to shearwalls with stucco finish over plywood.)

The second prediction to be made is the degree of drift at which initial connection fracture occurs. Under case A, the maximum connector deformation $\delta = Db/h$. Let δ_u denote the deformation capacity of a connection. Consider the value of D associated with failure, given case A and δ_u. The performance function for case A is given by Equation E-4. Failure occurs when $g_A < 0$, i.e., when $D > \delta_u h/b$. The performance function for case B is given by Equation E-5. Failure occurs when $D > \delta_u$. When $h/b = 2$, case A allows for twice the level of drift before the first connection fails. The hypothesis can be tested by comparing drift at the strength limit state (denoted here by D_{SLS}) with $\delta_u h/b$, requiring knowledge of δ_u.

$$g_A(\delta, \delta_u) = \delta_u - \delta$$
$$= \delta_u - Db/h \quad \text{(E-4)}$$
$$g_B(\delta, \delta_u) = \delta_u - \delta$$
$$= \delta_u - D \quad \text{(E-5)}$$

Fonseca *et al.* (in progress) are performing laboratory tests of a variety of wood connections; their data on δ_u are not yet available. However, Rosowsky *et al.* (2001) present some preliminary results of the Fonseca *et al.* (in progress) tests, as well as tests by Durham (1998) and Dolan (1989). Test results are summarized in Table E-2. Pardoen *et al.* (in progress) provide test results on drift associated with shearwalls of construction similar to the materials of the Dolan (1989) test: Pardoen *et al.*'s groups 1, 2, and 5 all represent 3/8-in. STR1 plywood connected to Douglas fir framing with 8d common nails. Group 10 is similar to the Fonseca *et al.* (in progress) tests, although the former uses 7/16-in. OSB; the latter, 3/8-in. Recalling that drift associated with deformation of anchorage and sill connectors amount to 1.5 to 2 times that associated with framing-nail deformation (per Uang *et al.,* 2001) one can compare the predicted drift at first connection failure (denoted by $D_{LS1} = 2.75\ \delta_u h/b$) with observed D_{SLS} from the Pardoen *et al.* (in progress) tests. Predicted and observed drifts at LS1 and SLS are shown in Table E-3 and plotted in Figure E-5. In the figure, dots represent mean values; error bars, plus and minus one standard deviation. Although it is a very small test sample, it tends to support the hypothesis.

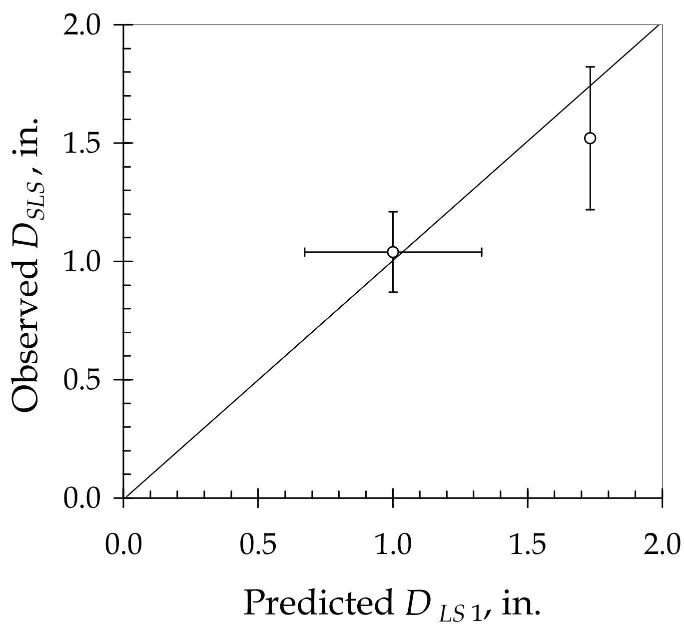
Table E-2:
Connection Ultimate Capacity.

	Sheathing	Framing	Connector	δ_u (in.)
Durham (1998)	3/8" OSB 1	SPF	50-mm power-driven spiral nails	0.492
Fonseca *et al.* (in progress)	3/8" OSB 2	DFL	8d common nails	0.250
Fonseca *et al.* (in progress)	3/8" OSB MFG1 2	DFL	8d common nails	0.138
Fonseca *et al.* (in progress)	3/8" OSB MFG2 2	DFL	8d common nails	0.157
Dolan (1989)	3/8" plywood 3	SPF	8d common nails	0.315

Table E-3:
Comparison of Limit State 1 with Pardoen *et al.* (In Progress) Strength Limit State.

Connector test	D_{LS1}: mean, sample standard deviation (in.)	Shearwall test ID (Pardoen *et al.* group)	D_{SLS}: mean, sample standard deviation (in.)
Fonseca *et al.* (in progress)	1.00, 0.33	10	1.04, 0.18
Dolan (1989)	1.73, --	1, 2, 5	1.52, 0.30

Figure E-5:
Support for Hypothesis That Maximum Strength is Associated with First Connector Failure.



Consider now LS2, the point at which the connection failure in the wall is extensive enough to cause the wall to be unstable, i.e., connections fail on three edges of each sheet by fracture, pullout, or tearthrough. It will be assumed that such extensive damage to the connections is also associated with extensive damage to the sheet, and possibly to the studs. Repair would entail replacing a heavily damaged sheet and any split studs. Neither the Pardoen *et al.* (in progress) tests nor the Uang *et al.* (2001) tests identify a point at which this state occurs. At least until test data provide the necessary damage information the collapse limit state (CLS) can be used as a proxy for LS2. In conclusion, for unfinished structural sheathing, limit state 1 (structural sheathing connection failure) will be associated here with the strength limit state (LS1 = SLS), while limit state 2 (collapse) with the collapse limit state (LS2 = CLS).

Drift levels associated with these two limit states are drawn from Pardoen *et al.* (in progress), tests 1, 2, 5, and 25 (3/8" STR-1 plywood), tests 3, 4, 9 and 26 (15/32" STR-1 plywood), test group 10 (7/16" OSB), and test groups 11, 12, and 13 (15/32" OSB). Let m_i denote the mean interstory drift associated with a limit state for test group *I*, and let s_i denote the standard deviation of interstory drift associated with the limit state for test group *i*. Let *m* denote the mean interstory drift for all samples in *n* test groups. Let all test groups have the same number of samples per group. Let *s* denote the standard deviation of drift associated with all the samples in the *n* test groups. The value of *m* is then given by Equation E-6, and *s* is given by Equation E-7. If the drift associated with reaching a limit state is lognormally distributed, then the median value of the distribution (denoted by x_m) is given by Equation E-8, and the logarithmic standard deviation (denoted by β) is given by Equation E-9. Using these equations and the statistics for the test groups listed above, Table E-4 gives the capacities of the unfinished shearwall assemblies considered here.

$$m = \frac{1}{n}\sum_i m_i \tag{E-6}$$

$$s = \sqrt{\sum_i s_i^2 + \sum_i \left(m - m_i\right)^2} \tag{E-7}$$

$$x_m = \frac{m}{\sqrt{1 + \left(\frac{s}{m}\right)^2}} \tag{E-8}$$

$$\beta = \sqrt{ln\left(1 + \left(\frac{s}{m}\right)^2\right)} \tag{E-9}$$

Table E-4. Fragility parameters of unfinished structural shearwalls.

Sheathing	Pardoen *et al.* test groups	LS1 (SLS) m	s	x_m	β	LS2 (CLS) m	s	x_m	β
3/8 STR 1	1	0.014	0.001	0.014	0.10	0.022	0.000	0.022	0.01
3/8 STR 1	2	0.019	0.003	0.019	0.14	0.024	0.000	0.024	0.01
3/8 STR 1	5	0.013	0.001	0.013	0.06	0.020	0.005	0.019	0.25
3/8 STR 1	25	0.016	0.002	0.016	0.10	0.034	0.001	0.034	0.02
3/8 STR 1	1, 2, 5, & 25	0.016	0.006	0.015	0.36	0.025	0.012	0.023	0.45
15/32 STR 1	3	0.018	0.003	0.018	0.18	0.030	0.000	0.030	0.01
15/32 STR 1	4	0.020	0.002	0.020	0.11	0.028	0.001	0.028	0.03
15/32 STR 1	9	0.020	0.001	0.020	0.06	0.031	0.007	0.030	0.21
15/32 STR 1	26	0.013	0.001	0.013	0.06	0.016	0.001	0.016	0.05
15/32 STR 1	3, 4, 9, & 26	0.017	0.007	0.016	0.39	0.026	0.014	0.023	0.50
7/16 OSB	10	0.010	0.002	0.010	0.17	0.022	0.003	0.022	0.13
15/32 OSB	11	0.010	0.001	0.009	0.12	0.020	0.001	0.020	0.05
15/32 OSB	12	0.014	0.001	0.014	0.09	0.025	0.001	0.025	0.03
15/32 OSB	13	0.014	0.001	0.014	0.09	0.025	0.013	0.022	0.49
15/32 OSB	11, 12, & 13	0.013	0.004	0.012	0.33	0.023	0.013	0.020	0.54

Plywood and Oriented Strandboard (OSB) Sheathing, With Stucco Finish

In cases where stucco is applied over the structural sheathing, the stucco inhibits relative slip of adjacent pieces of sheathing along the vertical joint between them, and case A of Figure E-4 no longer governs behavior, but rather case B.

Uang *et al.* (2001) observe from their two tests of stucco-finished shearwalls (one with plywood sheathing, one with OSB) that "the failure mode switched from being dominated by the fasteners in the specimens without stucco to failure of the framing members at the bottom portion of the shearwalls in the specimens with stucco." Photos and diagrams of these tests show that by the end of the test the bottom edge of the sheathing and stucco had become disconnected from the sill plate, and the studs separated from the sill by fracture of the studs at their connectors. It seems reasonable to conclude that the sheathing-to-sill connections failed first and all at approximately the same drift level, causing all the racking load to be transferred through the stud-to-sill connections, which consequently failed.

One would expect failure in the stucco-finished shearwall (case B) to occur when $D \approx 2.75\delta_u$, but no data on δ_u are available yet for the same connectors used by Uang *et al.* (2001), so this hypothesis cannot be tested. Note however that the Uang *et al.* (2001) tests experienced SLS at 2.0% and 3.0% drift (**Figure E-6** and **Figure E-7**) approximately half that experienced by their test samples that had no stucco, as shown in the two figures.

Based on these observations, three limit states will be considered.

LS1 Fracture of sheathing-to-framing connections. Repair requires chipping away of stucco at the fractured edge, re-nailing of the structural sheathing, reinstalling the chipped-away stucco, and painting. Occurs at approximately half the drift associated with shearwalls without stucco finish.

LS2 Collapse. Occurs at approximately half the drift associated unfinished shearwalls without stucco finish.

LS3 Cracking of stucco at reentrant corners. Repair requires patching cracks and repainting the wall. Based on the Chai *et al.* (2001) data, this limit state occurs at peak transient drift with median x_m = 1.2% and logarithmic standard deviation of 0.48. Since this drift is typically in excess of that causing LS1 to be exceeded, and in many cases LS2, repairs for LS3 would be redundant. Thus, one can ignore LS3 for present purposes.

Table E-5:
Fragility Parameters of Stucco-Finished Structural Shearwalls.

Sheathing	Pardoen *et al.* test groups	LS1 (SLS) x_m	LS1 (SLS) β	LS2 (CLS) x_m	LS2 (CLS) β
3/8 STR 1	1, 2, 5, & 25	0.007	0.36	0.011	0.45
7/16 OSB	10	0.005	0.17	0.011	0.13
15/32 OSB	11, 12, & 13	0.006	0.33	0.010	0.54
15/32 STR 1	3, 4, 9, & 26	0.008	0.39	0.011	0.50

Figure E-6:
Load-Displacement Behavior of Stucco-Finished OSB Shearwall (Uang *et al.*, 2001).

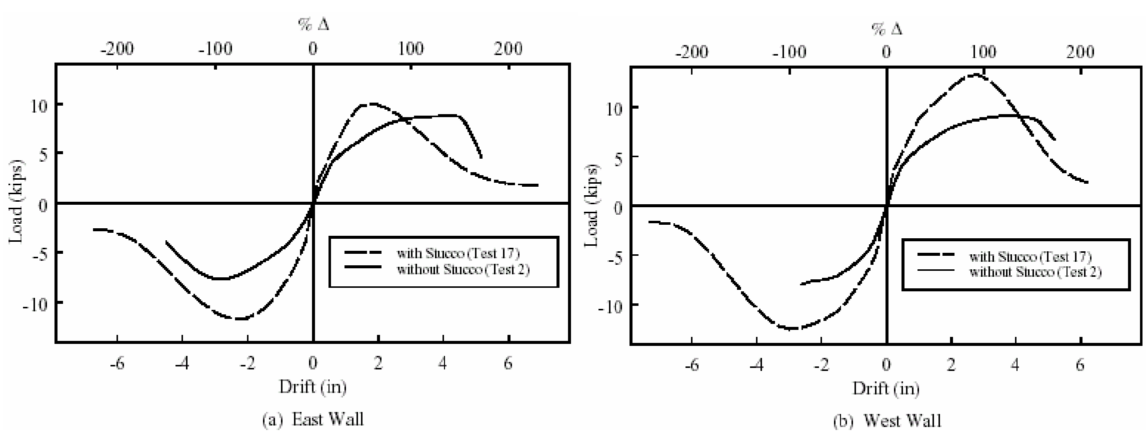
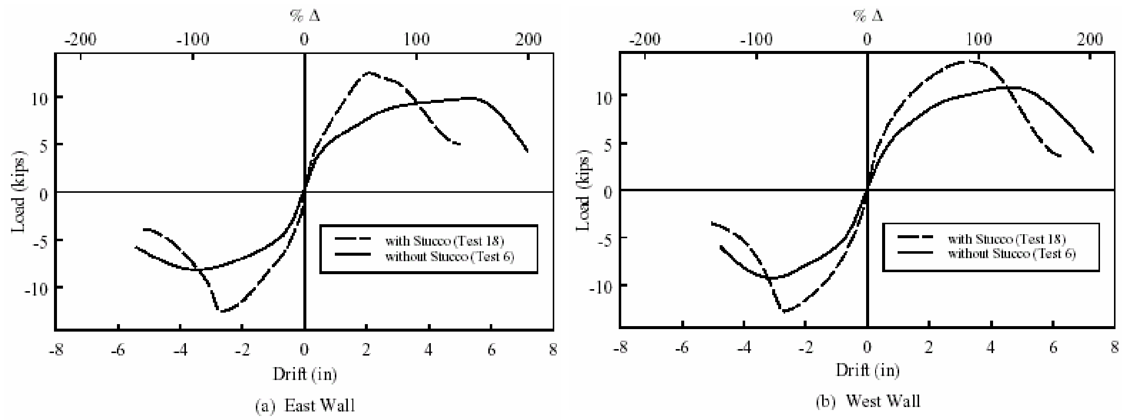


Figure E-7:
Load-Displacement Behavior Of Stucco-Finished Plywood Shearwall (Uang *et al.*, 2001).



Windows

A modest amount of empirical testing has been performed on the seismic resistance of windows. Pantelides *et al.* (1994), and Behr *et al.* (1995, 1998) performed laboratory tests on curtain-wall glazing subject to racking as in earthquake. The most recent study, Behr *et al.* (1998) is the most applicable for present purposes. The researchers performed displacement-controlled cyclic load tests on several dozen samples of 5-ft by 6-ft architectural glass assemblies (lites in aluminum extruded frames with standard mounting of lite in frame). Of the limit states examined, the most relevant for present purposes is visible cracking of glass. The investigators reports mean and standard deviation of in-plane interstory drift index associated with cracking, for 14 different combinations of framing system, glass type, and sealant. The test data were analyzed by Porter (2000), in which it was found that a lognormal distribution on capacity fit the data well. The resulting parameters of the lognormal distribution are: median capacity, denoted by x_m, is 0.040, and the logarithmic standard deviation of capacity, denoted by β, is 0.366.

Sucuoglu *et al.* (1997) present an analytical approach to estimating the drift capacity of window glass. They examine two failure modes: cracking because of in-plane deformation and because of out-of-plane vibration. They report that observations from past earthquakes indicate that in-plane deformation is the primary cause of glass breakage. In a model proposed by the authors, the capacity of glass to resist fracture comes from two sources: the drift accommodated by the gap between the frame and the edge of the glass, and from the in-plane diagonal shortening of the glass resulting from out-of-plane buckling. They provide a theoretical equation for the the drift ratio capacity, i.e., the amount of transient drift angle that the window can accommodate without fracturing. The glass fractures when there is no more gap between the frame and the pane, and the tensile stress in the deformed glass reaches the tensile capacity. The authors examined a variety of glass dimensions and provided sample drift capacity values for these systems, ranging between 2.2% and 3.6% drift, somewhat lower than the results of the Behr *et al.* (1998) laboratory tests.

ATC-38 (Applied Technology Council, in press) provides a source of empirical damage data from an extensive damage survey performed after the Northridge Earthquake. Structural engineers surveyed 199 woodframe buildings located near strong-motion instruments in the Los Angeles area. Surveyors estimated the fraction of windows damaged at each building. Along with damage data, physical characteristics of the buildings are also recorded, most notably height, and the strong-motion instrument closest to the building. Using height as an indicator of fundamental period, together with spectral displacement at that fundamental period, it is possible to estimate the peak transient drift ratio at each building. The data were analyzed, but it was found that they do not show a trend, i.e., no apparent relationship between drift and probability of cracking.

The lack of a trend relating window damage in Northridge to estimated peak transient drift ratio has two possible explanations: either no relationship exists between drift ratio and window cracking, or at these levels of drift, statistical noise overwhelms whatever trend does exist. Possible sources of statistical noise include variability in soil conditions between buildings, variability of true fundamental building period relative to the one assumed based on building height, variability of window materials and installation details, uncertainty on the part of surveyors regarding the meaning of the survey questions, and approximations made by the surveyors in estimating degree of damage. Regardless of the apparent lack of a trend, two observations can be made from the data. First, the data do not strongly conflict with the Behr *et al.* (1998) tests. The laboratory data show that the tested samples did not crack until drift exceeded 2.5%. Only seven structures in the Northridge sample of woodframe buildings experienced drifts in that range (calculated as described above). Second, window damage in woodframe structures subjected to strong shaking are not necessarily widespread. Of the buildings surveyed, only 1 in 12 had any window breakage, although among these, on average 1 in 4 windows was broken.

For the present study then, the analytical model proposed by Suculoglu *et al.* (1997) appears to be most useful. Uncertainty in gap size and glass material properties can be used to create a theoretical probability distribution on window capacity. The wood-framed windows of the small house have pane dimensions of approximately 30” x 20” x 1/8”. Three variables are taken as random: the gap between the edge of the glass and the frame, denoted by c, is taken as having a median value of 3/16” and a logarithmic standard deviation of 0.3. The modulus of elasticity of the glass, denoted by E, is taken as having a median value of 17,000 ksi and a logarithmic standard deviation 0.05. The tensile strength, denoted by σ, is taken as having a median value of 6 ksi with a logarithmic standard deviation of 0.3. These assumptions result in a glass cracking capacity with a median value of 0.030 and a logarithmic standard deviation of 0.29. That is to say, 50% of windows of these dimensions, when subjected to a drift angle of 3%, will crack. Thus, the analytical approach to the drift capacity of the glazing in the small house implies that it cracks at drifts somewhat lower than the commercial glazing system tested by Behr *et al.* (1998), i.e., at 3% drift rather than 4%. The uncertainty on drift capacity between the two approaches agrees.

Water Heater

Water heaters that are not strapped to the wall tend to overturn in earthquakes, and often their relatively slender legs buckle. A simple static analysis was performed to estimate the probability of overturning, based on the height and diameter of the water tank, the empty weight of the tank and the weight of the water in the tank. The tank dimensions, empty weight of the tank, and the height of the legs were taken as uncertain, and a simulation was performed to determine the distribution of acceleration necessary to cause the tank to overturn. The diameter was taken to be 18 in. ± 6 in. (i.e., 6 in. standard deviation, normal distribution). The tank height was taken to be 48 in. ± 12 in, the height of the legs 12 in. ± 6 in., and the empty weight of the heater taken to be 50 lb ± 15 lb. These assumptions result in an estimate of the median base acceleration causing overturning, denoted by x_m, of 0.61g, (i.e., 0.61 times the acceleration due to gravity) with a logarithmic standard deviation, denoted by β, of 0.37.

A more sophisticated analysis by Ishiyama (1983) confirms the overturning accelerations indicated by this simplistic analysis, but indicates two additional parameters, the diaphragm velocity and displacement, also strongly affect the potential for overturning. A three-parameter capacity criterion could theoretically be incorporated into the ABV, but for the present the single parameter, acceleration, will be used.

Finally, Makris and Konstantinidis (2001) propose a radically different approach to calculating the fragility of rigid blocks subjected to overturning from seismic excitation, but these procedures were unavailable at the time of this writing. The validity of the foregoing theoretical fragility function should be revisited in the light of this new study.

Summary of Assembly Fragilities

Table E-6 summarizes the assembly fragility parameters used in the present project. Each row reflects one fragility function. It shows an assembly type and limit state, the structural response to which the assembly is most sensitive, and the median and logarithmic standard deviation of the capacity, when modeled as a lognormal cumulative distribution. In the response column, PTD refers to the unitless peak transient drift ratio and PDA refers to peak diaphragm acceleration, in units of gravities.

Table E-6:
Summary of Assembly Fragility Parameters

Assembly Type,	Description	d	Limit State	Response	x_m	β
4.5.110.2101.01	Exterior shearwall, 3/8 C-D ply, 2x4, 16" OC, 7/8" stucco ext, no int finish	1	Fracture of sheathing-to-framing connections	PTD	0.007	0.37
		2	Collapse	PTD	0.011	0.45
4.5.110.2101.02	Exterior shearwall, 15/32 C-D ply, 2x4, 16" OC, 7/8" stucco ext, no int finish	1	Fracture of sheathing-to-framing connections	PTD	0.010	0.3
		2	Collapse	PTD	0.015	0.5
4.5.110.2111.01	Exterior shearwall, 7/16 OSB, 2x4, 16" OC, 7/8" stucco ext, no int finish	1	Fracture of sheathing-to-framing connections	PTD	0.005	0.3
		2	Collapse	PTD	0.011	0.5
4.5.110.2501.01	Exterior wall, no structural sheathing, 2x4, 16" OC, 7/8" stucco ext, no int finish	1	Cracking of stucco	PTD	0.0076	0.46
		2	Heavy damage to stucco and framing	PTD	0.031	0.47
4.6.152.1700.01	Doors, sliding, patio, aluminum, standard, 6'-0"x6'-8", with wood frame, insulated glass	1	Cracked	PTD	0.028	0.44
4.7.110.6600.01	Window, Al frame, sliding, standard glass, pane <25 sf	1	Cracked	PTD	0.030	0.4
4.7.110.6609.01	Window, Al frame, fixed, standard glass, 80"x80" pane	1	Cracked	PTD	0.030	0.3
4.7.100.3001.01	Windows, wood, double hung, standard glass, 3'x4'	1	Cracked	PTD	0.030	0.29
6.1.510.1202.01	GWB partition, no structural sheathing, ½" GWB one side, 2x4, 16" OC	1	Paint damage at connector heads, cracks at opening	PTD	0.0025	0.5
		2	Fastener tearthrough, fracture at taped joints	PTD	0.0040	0.5
		3	Collapse	PTD	0.016	0.2
6.1.510.1203.01	GWB finish, ½", one side, on 2x4, 16"OC	1	Paint damage at connector heads, cracks at opening	PTD	0.0025	0.5
		2	Fastener tearthrough, fracture at taped joints	PTD	0.0040	0.5
		3	Collapse	PTD	0.016	0.2

Table E-6:
Summary of Assembly Fragility Parameters (Cont.)

Assembly Type, Description		d	Limit State	Response	x_m	β
6.1.520.1201.01	Interior shearwall, 3/8 C-D ply, 2x4, 16" OC, 1/2" GWB finish one side	1	Paint damage at connector heads, cracks at opening	PTD	0.0025	0.5
		2	Fracture of sheathing-to-framing connections	PTD	0.014	0.37
		3	Collapse	PTD	0.023	0.45
6.1.520.1201.02	Interior shearwall, 15/32 C-D ply, 2x4, 16" OC, 1/2" GWB finish one side	1	Paint damage at connector heads, cracks at opening	PTD	0.0025	0.5
		2	Fracture of sheathing-to-framing connections	PTD	0.019	0.3
		3	Collapse	PTD	0.029	0.5
6.1.520.1202.01	Interior sheathing, 3/8 C-D ply, 1/2" GWB finish one side, on 2x4 16" OC	1	Paint damage at connector heads, cracks at opening	PTD	0.0025	0.5
		2	Fracture of sheathing-to-framing connections	PTD	0.014	0.37
		3	Collapse	PTD	0.023	0.45
6.1.520.1202.02	Interior sheathing, 15/32 C-D ply, 1/2" GWB finish one side, on 2x4, 16" OC	1	Paint damage at connector heads, cracks at opening	PTD	0.0025	0.5
		2	Fracture of sheathing-to-framing connections	PTD	0.019	0.3
		3	Collapse	PTD	0.029	0.5
6.1.520.1211.01	Interior shearwall, 7/16 OSB, 2x4, 16" OC, 1/2" GWB finish one side	1	Paint damage at connector heads, cracks at opening	PTD	0.0025	0.5
		2	Fracture of sheathing-to-framing connections	PTD	0.010	0.3
		3	Collapse	PTD	0.022	0.5
6.1.520.1212.01	Interior sheathing, 7/16 OSB, 1/2" GWB finish one side, on 2x4 16" OC	1	Paint damage at connector heads, cracks at opening	PTD	0.0025	0.5
		2	Fracture of sheathing-to-framing connections	PTD	0.010	0.3
		3	Collapse	PTD	0.022	0.5
8.1.160.1820.01	Electric water heater, residential, 100F rise, 50 gal, 9 kW 37 GPH	1	Overturned	PDA	0.61	0.37

Appendix F. Building Construction Cost and Damage Estimation

Introduction

Post-earthquake evaluation of building damage and necessary repairs can involve working under chaotic conditions. Natural or manmade disasters may bring devastating losses to thousands of property owners. Lives are interrupted, companies lose money, and quick relief is needed. It is imperative that a detailed and accurate estimate of the cost to repair damage to buildings be completed as soon as possible so the restoration process can begin.

Post-earthquake evaluation of building damage and loss presents a unique challenge: the extent of damage is not always clear and desired repair of any damage may depend on who is paying the repair costs. Also, insurance policies are different for earthquake damage than for normal casualty type loses. Deductibles are usually based on a substantial percentage (5% to 25%) of the replacement cost of the building. For example, if the cost to rebuild a house is $300,000, the deductible could be $15,000 to $75,000, depending on the policy. Some items such as sidewalks and driveways, pools and masonry are excluded from coverage.

Based on these shortfalls in insurance coverage, some insureds become somewhat creative in the scope of damages claimed. However, the insurer's responsibility is to pay only what is due under the terms of the policy. Needless to say, this can lead to many different opinions of what work is actually required. It is important for an estimator working on an earthquake repair to be diligent in verifying all possible damage, researching pre-existing conditions, and communicating with the insured and insurer to be sure to know their expectations.

In theory, estimating the cost to repair a damaged building is a simple process: determine the quantities of materials and labor needed, multiply those quantities by their prevailing unit costs, add soft costs, total, and prepare a report. When the scope of necessary repair work is well defined, prices are stable, and the estimating objective is a competitive bid, cost estimates prepared by different, experienced estimators will generally be comparable and predictable. The scope of an estimate varies according to the interest of the client. Damage-repair estimates and databases vary and may not be consistent.

A major disaster such as the Northridge Earthquake greatly alters the estimating environment: the scope of necessary repair work is not well defined, unit prices can become inflated because of scarcity of labor and materials, and many estimates are prepared with an eye toward maximizing financial recovery rather than actually completing the work. The result is both increased cost estimates and increased variation between estimates. This uncertainty makes reliable prediction of losses in future earthquakes extremely difficult, especially in the context of earthquake insurance.

Repair-cost estimates may be prepared by various groups for various purposes. In the course of preparing competitive bids for repair work, contractors estimate the cost of the work being pursued as a basis for their bid. Competitive bids generally result in the lowest cost. Insurance companies (and to a certain extent the Federal Emergency Management Agency and the Small Business Administration) employ adjusters to determine the scope and costs of repair work. The adjuster can perform this work on his or her own, request a contractor to bid the project or hire consultants to provide estimates and reports on the structural and mechanical integrity of the damaged buildings. Building owners, either on their own or with the assistance of a public adjustor, can also hire contractors or consultants to provide repair cost estimates for use in negotiation with the insurer.

The following sections discuss each of the major aspects of repair-cost estimating and the unique challenges associated with estimating the costs of earthquake damage repair.

Scope of Repair

The basis of every repair cost estimate is a scope of work: what needs to be done to repair damage to a building caused by the earthquake. While straightforward in concept, this essential step can be the source of much disagreement. (This disagreement is the motivation for the development of guidelines for earthquake insurance-claims adjustment currently underway by the California Earthquake Authority.) Disagreements regarding scope of repair arise primarily in four areas:

- *Definition of damage*. Upon scrutiny, numerous imperfections can exist in any building. Lumber, for example, warps, checks, and splits as it dries. Does this constitute damage or merely the normal state of the construction material?
- *Causation of damage*. Buildings are subject to numerous damaging events ranging from soil settlement to water leaks. For example, unbeknownst to the owner, foundations of many buildings in California have been damaged by long-term soil settlement. If this damage is discovered following an earthquake, should it be included in the repair cost estimate?
- *Presence of hidden damage*. Many of the structural components of a building are hidden from view, and in woodframe dwellings, many materials, such as plaster or drywall finish materials, may not have been designed as structural elements but can significantly affect earthquake performance and require evaluation from a structural point of view. What allowance should the estimator make for inspection for or repair of hidden damage? Two engineers can differ in their opinions as to how much residual earthquake resistance a building has, given its severity of shaking and any damage experienced in an initial earthquake. The process of making a building whole again theoretically implies that any repairs made will enable the building to experience the same earthquake while experiencing exactly the damage that resulted the first time: if less damage resulted, this theoretical test would indicate that the repair was actually an upgrade; if more damage resulted, this would indicate inadequate repair.
- *Repair of damage*. There are many methods available for repair of damage in buildings. Cracks in a concrete foundation, for example, can be repaired by injection of a structural epoxy, installation of steel plate, or removal and replacement. Which repair is appropriate?

Answers to these and related questions can have an enormous impact on repair cost estimates. The estimator may have the benefit of reports from other consultants (structural, geotechnical, environmental, architectural) to guide the development of the repair scope or the burden may fall to the estimator alone.

Development of the scope of work begins with a site visit, once a building can be safely accessed. Site visits are essential if the extent of the damage is to be accurately determined. In addition to the estimator's visual inspection, tests may be needed to determine the final scope of repairs. Examples of such tests include environmental consultants to determine hazardous materials such as asbestos, lead paint or mold and destructive testing by the structural engineer to determine possible hidden damage.

The damage estimate should cover all elements that are permanently attached to the building. The estimate should not include contents such as washers, dryers, refrigerators (unless they are part of the building as in apartments), paintings, lamps, furnishings, area rugs, etc.

In developing the estimate, each room, office, or work area should be visually assessed. Photographs should be taken, measurements made, finishes noted, and the site's general condition described.

The estimator should write down all the finishes required for renovation. Interior examples are paint, drywall, flooring, wall covering, doors, finish hardware and cabinets. Exterior examples include plaster, siding, roofing, trim and site work. All these details are required in order to estimate the cost to replace or match the existing materials. Diligently and methodically logging information on an initial site visit can save hours of travel time on additional trips.

Importance of photographs. A complete photographic record of damage is a very important tool for documenting existing finishes and estimating replacement costs. After a structure is demolished, the estimator loses the chance to know exactly what products and materials were present.

A caution about blueprints. Blueprints can help, but are typically unavailable. When they are available, an estimator should keep in mind that what was actually built or installed may differ from the original plans.

Skill requirements. The skill level needed to make repairs also may be determined at the site. As an example, a smaller residential project may require a carpenter to perform demolition and some electrical work, frame walls, set doors and toilets, and paint.

Line of sight. When portions of a building are damaged, repairs are confined to that specific area in most cases. However, line-of-sight requirements may determine additional repairs and their costs. To illustrate: suppose a building has damaged carpet or wallpaper in one room that is connected by those same finishes to other rooms. An estimator learns that the carpet or wallpaper cannot be matched because it has been discontinued or would be from a different dye lot. That can drive a decision to replace all carpeting or wallpaper so that contiguous rooms will match. If one elevation of a building is damaged and needs painting, the entire building may need to be painted to match.

Code upgrades. Improvements mandated by legislative or regulatory rules relating to construction need to be considered by the estimator. Depending upon the extent of damage and when a structure was built, compliance with certain code requirements may be required as part of the repair. These can result from legislation such as the Americans with Disabilities Act, seismic or structural code changes, or mechanical and electrical updates. Even though required as part of the repair, code upgrades may not be covered by the property insurance and may need to be identified or segregated in the estimate.

Like kind and quality. Most insurance policies stipulate doing so "in like kind and quality." This means using in the cost estimate and subsequent repair similar, if not the same, types of products, finishes and construction elements. The estimate and the cost of final repairs depend on this assumption.

Costing

Once the scope has been determined, the estimator can begin estimating costs. Job costs to be considered by the estimator include: all labor and materials, subcontractor costs, job fees, taxes and insurance, permit fees, and bonds.

The estimator must translate all detail and measurements into square feet, lineal feet, square yards, cubic yards, cubic feet, lump sum, or as an allowance sum. In so doing, the estimator considers:

Quantities. Accurate quantities are critical to the success of any estimate. No matter how accurate unit cost data may be, no estimate is reliable if a mistake is made in the quantity of a material. Appropriate waste factors should be included in the quantities or adjusted in the unit cost.

Unit costs. Unit costs are the cost to install a single quantity of a specific measurement, such as a square foot of drywall. As an example, if a job requires 1,000 square feet of drywall, and the unit cost is $1.35, then the total cost is $1,350. Each unit cost should already include the subcontractor's mark-up but not the general contractor's overhead and profit.

The size, scope of work, and type of construction project significantly impact costs. Estimating new construction is made easier because accurate blueprints are usually available showing details and specifying products.

Retrofit construction or damage-repair estimates are much more difficult to calculate. Beyond the obvious costs (namely, material and labor) required for renovation, other variables affect any cost estimate. These include, but are not limited, to:

Labor rates. One variable that affects costs is the potential difficulty in finding skilled workers. Training, efficient crews, employee retention, and job management are growing challenges in the industry. Labor unions may require, as an example, the use of union laborers at a labor rate 20 to 40 percent higher than non-union laborers.

Spot material shortages. At the time of a disaster in a particular geographic area, there may be shortages of specific materials, such as lumber, roofing materials, drywall, etc. If this is the case, an estimator must factor in the cost of locating and shipping such materials.

Site conditions. Repairs may require working around an existing building's contents. There may be access or hillside challenges, out-of-plumb or out-of-square walls, outdated materials, or existing conditions that violate current building codes. Such items are all difficult to estimate because there may not be an exact unit cost in a database. Walking 20 extra feet to set up scaffolding on a hillside would be difficult to estimate as a unit cost, for example. Careful consideration must be given to account for additional labor-hours required to accomplish such tasks. Other examples may be taking 15 to 20 minutes longer to install a door in an out-of-square opening, or extra time to install stairs or wall framing when they have to be spliced into existing work. Other work that may take more time includes reinforcing or shoring, extra trips to find matching materials, unexpected problems, and possible restrictions in established residential or commercial areas.

Location. Project location is another factor. In dense urban areas, traffic and onsite storage limitations can increase costs. Outside a 20- to 25-mile radius of large cities, extra trucking costs may be required, and transportation costs can increase the cost of material. Labor rates vary depending on the location such as a rural area or an urban area.

Lumber. An example of how environmental conditions and inflationary pricing can affect estimates is the cost of lumber. Because lumber is a fairly volatile market, estimators must be careful about bids on it. For instance, some lumber companies will not commit to a price until construction actually begins. In response, the estimator must build in to the estimate, which has to be completed before the project starts, a potential rise in the market price of the lumber. There are no set percentages to use in anticipation of swings in the price of lumber. Experience and judgment have to be relied upon.

Small jobs. Small renovation jobs present unique challenges. For example, an estimator in such a case must realize that, for any particular project, skilled workers might participate in all construction phases. Unit costs can often run much higher for small projects. Costs for projects of substantially different size or type should be adjusted accordingly.

Direct overhead. Direct overhead costs are also known as general conditions. These can include a jobsite superintendent, vehicle allowance and fuel, temporary utilities, jobsite toilets, office trailers and supplies, storage sheds, jobsite cleanup, jobsite signs, small tools, first-aid equipment and temporary fencing. These items are usually listed separately from the items shown in the scope of repairs and are driven by the duration of the project. However, some computer software programs include these in their unit costs for each item. It is essential to be sure that these costs are not duplicated if one is using a software cost database.

Mark-ups. Most estimates must include a mark-up, that is, the amount added to an estimate after all job costs have been determined. The mark-up is designed to cover such non-job-related costs as office rent, office supplies, utilities, advertising, office salaries and professional fees that are not part of the estimate itself, plus a profit margin. The amount of mark-up varies in the industry, but a typical amount is 10 percent for indirect costs and 10 percent for profit, for a total of 20 percent. The profit mark-up usually decreases for larger projects.

Fees, taxes and insurance. Estimates for items such as permit fees, local sales tax and job-related insurance are generally handled as a lump-sum amount or a percentage of the project costs.

Miscellaneous. Other factors an estimator should consider include seasonal and weather conditions, production capabilities, adequate electricity, and safety requirements. For example, when a carpenter must install roof sheathing using a harness and cable, it reduces the carpenter's mobility and thus his or her production output. Other questions or decisions include: How far to take repairs? Once construction begins, what additional repairs will be required? Will the materials match? And so on.

All the cost variables added into an estimate make it challenging to provide accurate pricing, particularly when a disaster creates greater demand for labor and material.

Reporting

When the estimator finally sits down to prepare the damage cost report, there is a great deal of information and data that must be processed into an understandable form. Fortunately, there are many software packages, books and estimating systems available to help.

Such programs and resources may provide subcontractor costs, area modifications, unit costs for labor, materials and equipment, and a rule-of-thumb cost for most industry trades.

Some software packages also supply area modifications based on completed projects, taking into account many cost variables such as labor, materials, equipment needs, climate, job conditions and mark-up for a specific geographic area. These programs produce a national average and then reference specific U.S. cities, showing either an added or reduced cost percentage for each city. As examples, compared to the national average for construction costs, the San Diego area is currently 10 percent higher, the Los Angeles area is 14 percent higher, and the San Francisco area is 24 percent higher.

Two of the most commonly-used estimating software packages in the insurance industry are Xactimate and Boech. There are numerous other software programs that are designed for repair work. The common link to these programs is they scope by area and not by trade. Usually the program can tally the items for each trade. The difficulty with these programs is learning their codes for each item and trying to evaluate an estimate without always knowing how an item was coded. For example, one program may include wall texturing in drywall and another in painting.

While software programs are excellent tools to help produce an accurate and reliable estimate, the estimator must keep in mind that such estimations cannot account for all of the above-mentioned cost variables, and that peculiarities of individual projects can affect the true unit cost experienced. Thus, tabulated, generic unit costs are not guaranteed to fit every job.

Good estimates are tailored to each particular project. Using unit costs provided by an estimating program is no substitute for judgment, analysis, experience, skill, and knowledge.

Detail. In preparing the final estimate, there should be as much detail as possible. The more specifically the item is described, the more exact the prices can be. The more generally an item is described, the more assumptions must be made, resulting in less exact pricing.

In calculating costs without the use of estimating software, spreadsheet programs such as Microsoft Excel are essential. Many estimators, in fact, have template spreadsheets on hand that contain most building trade and general conditions items. The estimator simply inputs quantities and unit costs into a spreadsheet program, which automatically multiplies the quantity of measurement (square feet, lineal feet, etc.) by the unit cost. Usually this type of estimating is done by trade and not by area.

The Estimator

Both experience and knowledge affect the competency of an estimator. As an example, an estimator may have completed computer training and be very efficient in making spreadsheet calculations, yet may not have any construction experience. Conversely, another estimator may have a great deal of field experience, yet not be adept at contemporary software applications. This second estimator will view constructing the project in a different light because of hands-on experience. The most qualified estimator has credentials with equal portions of experience and knowledge.

The time it takes to compile estimates varies greatly. A small repair job may mean approximately four hours of time including travel. On the other hand, a larger project, comprising more rooms, offices or levels, may require two to three weeks to prepare. An estimator may be able to use his or her own database for cost purposes, but if he or she must consult with a subcontractor for some specialty items, it will take additional time. An estimator's fees depend upon the complexity and size of a project, and his or her time. Most general contractors do not charge for their estimates because they are trying to get the work. This cost becomes part of their non-job related overhead. If consultants are hired, they have no interest in the project and will charge for their time by the hour or a percentage of the project.

Each estimator views the same project differently. For example, one might take 10 identical blueprints of a home to 10 different estimators, and get 10 different bottom-line amounts to build it. Why? Because each estimator's assessment of quantity requirements, unit costs, project duration and mark-up differs. Also, less-experienced or less-knowledgeable estimators can make mistakes.

Estimating is part art and part science, with many variables that come into play. On many jobs, the range between high and low estimates can be 20 percent or more. There will always be legitimate disagreements on what the correct costs are, even when plans and specifications are available, and labor and material costs are identical. Estimators who can be relied upon to perform professional and accurate estimates with a mindset that they have a vested interest in the outcome are worth their weight in gold.

While the real world of disaster-loss estimation has many variables that must be taken into account, the estimating can still be a straightforward process provided the basics are followed. A successful project is built on the foundation of an accurate estimate. With these principles in mind, the costs to construction the index buildings, variants, retrofits, and redesigns are now presented, along with unit costs to perform detailed repairs.

Estimates For the Woodframe Building Project

Table F-1 summarizes estimates of the cost to construct the index buildings and their variants. The estimates assume that the buildings would be built in Santa Monica, California, using non-union labor, in the year 2001. Table F-2 details the cost to construct the small house (poor, typical, and superior-quality variants are assumed to cost the same to build.) Table F-3 summarizes the cost to building the large house. Table F-4 and Table F-5 summarize the cost to build the townhouse (all three units) and apartment building, respectively.

Figure F-1 presents the estimated cost to add structural sheathing to the unbraced cripple walls of the small house. The cost to construct the large house with the immediate-occupancy and rigid-diaphragm redesign measures is presented in Table F-6 and Table F-7, respectively. The cost of the limited–drift redesign measure for the townhouse is presented in Table F-8. Table F-9 and Table F-10 present the construction-cost estimates for the steel moment-frame and shearwall redesigns of the apartment building.

Unit costs to repair the various damageable assemblies are summarized in Table F-11 and detailed in a series of data sheets that follow the table. The table presents unit repair-cost estimates by assembly type and damage state, *D*. The table shows the nature of the repairs required to restore the damaged assembly, the unit by which assemblies are counted, the parameters of the repair-cost distribution (x_m and β) and a code to identify the data sheet that details the costs. (Under the framework used here, unit repair costs are assumed to be uncertain, distributed lognormally. The parameter x_m represents the median cost of the repair, and β represents the logarithmic standard deviation.)

Table F-1:
Estimates of the Cost to Construct Index Buildings, Retrofits, and Redesigns

Index Building	Variation	Cost of Construction ($)
Small house	Poor, typical, and superior quality	$136,641
Small house	Measure 1, braced cripple walls	$137,979
Large house	Poor, typical, and superior quality	$221,430
Large house	Measure 2, waist wall	$221,692
Large house	Measure 3, immediate occupancy	$228,919
Large house	Measure 4, rigid diaphragm	$221,702
Townhouse	Poor, typical, and superior quality	$497,582
Townhouse	Measure 5, limited drift	$499,280
Apartment	Poor, typical, and superior quality	$797,197
Apartment	Measure 7, steel moment frames	$826,201
Apartment	Measure 8, shearwalls	$808,524

Figure F-1:
Cost to Perform Small-House Index Building Retrofit

Description

Add new expansion anchors, replace existing anchor bolt washers with new 2”x2”x3/16” sq. washers, install new 2x4 blocking on top of existing sill plate as shear panel backing, install new 15/32” 5-ply plywood shear panel and nail as specified.

Labor Required

Expansion bolts: 8 ea. @ $20= $160
2”x2”x3/16” sq. washers: 31 ea. @ $3.50= $108.50
2x4 blocks: 42 ea. @ $4.75= $199.50
15/32” shear panel: 112 sf. @ $1.40= $156.80
Total Labor: $624.80

Materials and Equipment Required

Expansion bolts: 8 ea. @ $3.75= $30
2”x2”x3/16” sq. washers: 31 ea. @ $.88= $27.28
2x4 blocks: 42 ea. @ $.43= $18.06
15/32” shear panel: 112 sf. @ $.41= $45.92
Total Material: $121.26

Total Repair Cost and Duration (Mean, 90$^{\%ile}$, 10$^{\%ile}$)

Cost: Mean: $735	Upper bound: $870	Lower bound: $600
Duration: Mean: 15 mh	Upper bound: 18 mh	Lower bound: 12 mh

Comments, References

Labor costs are for 16 total mh @ $40 per hour for a non-union carpenter using nail guns and electric impact wrenches. Overhead and profit are excluded.

Table F-2:
Construction Cost Estimate, Small House

DIVISION	DESCRIPTION	DIVISION SUBTOTAL	DIVISION TOTAL
1	**GENERAL CONDITIONS**		**$26,144**
	Personnel	10,800	
	Small Tools	500	
	Temporary Facilities	1,688	
	Temporary Utilities	2,900	
	Clean – Up	5,256	
	Debris Removal	2,400	
	Testing & Inspections	2,600	
2	**SITEWORK**		**$8,050**
	Grading	8,050	
3	**CONCRETE**		**$4,930**
	Foundations	4,130	
	Pad Footings	800	
6	**CARPENTRY**		**$25,349**
	Rough Carpentry	19,347	
	Finish Carpentry	1,794	
	Cabinetry	4,208	
7	**MOISTURE PROTECTION**		**$3,638**
	Insulation	1,824	
	Roofing	1,814	
8	**DOORS, WINDOWS, & GLASS**		**$9,777**
	Wood Doors and Frames	2,765	
	Sliding Glass Door	736	
	Finish Hardware	790	
	Windows	5,486	
9	**FINISHES**		**$20,286**
	Lath & Plaster	3,543	
	Drywall	4,511	
	Ceramic Tile	2,341	
	Carpeting	2,771	
	Vinyl Flooring	452	
	Painting	6,667	
10	**SPECIALTIES**		**$196**
	Toilet Accessories	196	
11	**EQUIPMENT**		**$870**
	Appliances	870	
15	**MECHANICAL**		**$13,155**
	Plumbing	12,055	
	HVAC	1,100	
16	**ELECTRICAL**		**$5,540**
	Electrical Devices	5,540	
	SUBTOTAL	117,934	**117,934**
	Contractors overhead, profit, taxes, etc.		**17,823**
	TOTAL CONSTRUCTION		**135,757**
	Soft Costs		
1	**Building Permit (0.75%)**		**885**
	TOTAL		**136,641**

Table F-3:
Construction Cost Estimate, Large House

DIVISION	DESCRIPTION	DIVISION SUBTOTAL	DIVISION TOTAL
1	**GENERAL CONDITIONS**		**$81,430**
	Personnel	60,200	
	Small Tools	500	
	Temporary Facilities	3,000	
	Temporary Utilities	4,650	
	Clean - Up	7,280	
	Debris Removal	3,200	
	Testing & Inspections	2,600	
2	**SITEWORK**		**$7,000**
	Grading	7,000	
3	**CONCRETE**		**$12,275**
	Foundations	6,490	
	Slab on Grade	5,785	
6	**CARPENTRY**		**$89,202**
	Rough Carpentry	69,410	
	Finish Carpentry	7,552	
	Cabinetry	12,240	
7	**MOISTURE PROTECTION**		**$9,725**
	Insulation	3,495	
	Roofing	6,230	
8	**DOORS, WINDOWS, & GLASS**		**$14,788**
	Wood Doors and Frames	3,705	
	Sliding Glass Door	5,430	
	Garage Door	650	
	Finish Hardware	988	
	Windows	4,015	
9	**FINISHES**		**$38,212**
	Lath & Plaster	7,700	
	Drywall	11,234	
	Hardwood Flooring	579	
	Ceramic Tile	4,070	
	Carpeting	4,813	
	Painting	9,814	
10	**SPECIALTIES**		**$615**
	Toilet Accessories	615	
11	**EQUIPMENT**		**$1,720**
	Appliances	1,720	
15	**MECHANICAL**		**$25,347**
	Plumbing	18,952	
	HVAC	6,395	
16	**ELECTRICAL**		**$6,405**
	Electrical Devices	6,405	
	SUBTOTAL	286,719	**286,719**
	Contractors overhead, profit, taxes, etc.		**43,653**
	TOTAL CONSTRUCTION		**330,372**
	Soft Costs		
1	**Building Permit (1.5%)**		**4,301**
	TOTAL		**334,672**

Table F-4:
Construction Cost Estimate, Townhouse

DIVISION	DESCRIPTION	DIVISION SUBTOTAL	DIVISION TOTAL
1	**GENERAL CONDITIONS**		**$ 71,123**
	Personnel	48,000	
	Small Tools	500	
	Temporary Facilities	3,625	
	Temporary Utilities	4,650	
	Clean – Up	5,348	
	Debris Removal	6,400	
	Testing & Inspections	2,600	
2	**SITEWORK**		**$ 13,925**
	Grading	13,925	
3	**CONCRETE**		**$ 20,180**
	Foundations	9,260	
	Slab on Grade	10,920	
6	**CARPENTRY**		**$ 97,942**
	Rough Carpentry	72,016	
	Finish Carpentry	5,510	
	Cabinetry	20,416	
7	**MOISTURE PROTECTION**		**$ 22,406**
	Insulation	10,814	
	Roofing	11,593	
8	**DOORS, WINDOWS, & GLASS**		**$ 26,918**
	Wood Doors and Frames	6,678	
	Sliding Glass Door	6,375	
	Garage Door	2,181	
	Finish Hardware	2,375	
	Windows	9,309	
9	**FINISHES**		**$ 92,645**
	Lath & Plaster	19,384	
	Drywall	24,713	
	Ceramic Tile	13,744	
	Carpeting	8,906	
	Painting	25,897	
10	**SPECIALTIES**		**$ 3,200**
	Toilet Accessories	3,200	
11	**EQUIPMENT**		**$ 4,410**
	Appliances	4,410	
15	**MECHANICAL**		**$ 54,052**
	Plumbing	39,247	
	HVAC	14,805	
16	**ELECTRICAL**		**$ 19,485**
	Electrical Devices	19,485	
	SUBTOTAL	426,286	**426,286**
	Contractors overhead, profit, taxes, etc.		**64,902**
	TOTAL CONSTRUCTION		**$491,188**
	DESIGN		
1	**Building Permit**		**6,394**
	TOTAL		**$497,582**

Table F-5:
Construction Cost Estimate, Apartment Building

DIVISION	DESCRIPTION	DIVISION SUBTOTAL	DIVISION TOTAL
1	**GENERAL CONDITIONS**		**$ 103,189**
	Personnel	66,000	
	Small Tools	1,500	
	Temporary Facilities	4,456	
	Temporary Utilities	6,650	
	Clean - Up	13,982	
	Debris Removal	8,000	
	Testing & Inspections	2,600	
2	**SITEWORK**		**$ 14,083**
	Grading	14,083	
3	**CONCRETE**		**$ 25,071**
	Foundations	10,160	
	Pad Footings	14,911	
5	**METALS**		**$ 17,300**
	Structural Steel	17,300	
6	**CARPENTRY**		**$ 165,600**
	Rough Carpentry	135,416	
	Finish Carpentry	7,659	
	Cabinetry	22,525	
7	**MOISTURE PROTECTION**		**$ 15,879**
	Insulation	8,994	
	Roofing	6,885	
8	**DOORS, WINDOWS, & GLASS**		**$ 48,609**
	Wood Doors and Frames	13,060	
	Finish Hardware	9,143	
	Windows	26,406	
9	**FINISHES**		**$ 157,193**
	Lath & Plaster	35,569	
	Drywall	44,139	
	Ceramic Tile	14,640	
	Carpeting	15,725	
	Vinyl Flooring	3,677	
	Painting	43,443	
10	**SPECIALTIES**		**$ 1,955**
	Toilet Accessories	1,955	
11	**EQUIPMENT**		**$ 9,600**
	Appliances	9,600	
15	**MECHANICAL**		**$ 83,652**
	Plumbing	72,652	
	HVAC	11,000	
16	**ELECTRICAL**		**$ 40,840**
	Electrical Devices	40,840	
	SUBTOTAL	682,970	**682,970**
	Contractors overhead, profit, taxes, etc.		**103,982**
	TOTAL CONSTRUCTION		**$ 786,952**
	DESIGN		
1	**Building Permit (1.5%)**		**10,245**
	TOTAL		**$ 797,197**

Table F-6:
Construction Cost Estimate, Large House, Immediate-Occupancy Redesign

DIVISION	DESCRIPTION	DIVISION SUBTOTAL	DIVISION TOTAL
1	**GENERAL CONDITIONS**		**$ 36,793**
	Personnel	15,600	
	Small Tools	600	
	Temporary Facilities	2,438	
	Temporary Utilities	3,900	
	Clean – Up	7,655	
	Debris Removal	4,000	
	Testing & Inspections	2,600	
2	**SITEWORK**		**$ 9,500**
	Grading	9,500	
3	**CONCRETE**		**$ 12,275**
	Foundations	6,490	
	Slab on Grade	5,785	
6	**CARPENTRY**		**$ 46,933**
	Rough Carpentry	31,063	
	Finish Carpentry	2,671	
	Cabinetry	13,199	
7	**MOISTURE PROTECTION**		**$ 9,081**
	Insulation	3,617	
	Roofing	5,465	
8	**DOORS, WINDOWS, & GLASS**		**$ 9,930**
	Wood Doors and Frames	2,518	
	Sliding Glass Door	2,610	
	Garage Door	1,162	
	Finish Hardware	853	
	Windows	2,787	
9	**FINISHES**		**$ 38,760**
	Lath & Plaster	7,889	
	Drywall	9,060	
	Hardwood Flooring	882	
	Ceramic Tile	7,545	
	Carpeting	4,544	
	Painting	8,841	
10	**SPECIALTIES**		**$ 542**
	Toilet Accessories	542	
11	**EQUIPMENT**		**$ 1,470**
	Appliances	1,470	
15	**MECHANICAL**		**$ 17,682**
	Plumbing	13,487	
	HVAC	4,195	
16	**ELECTRICAL**		**$ 8,150**
	Electrical Devices	8,150	
	SUBTOTAL	191,114	**191,114**
	Permits		**1,433**
	Contractors overhead, profit, taxes, etc.		**28,882**
	TOTAL CONSTRUCTION		**221,430**

Table F-7:
Construction Cost Estimate, Large House, Rigid-Diaphragm Redesign

DIVISION	DESCRIPTION	DIVISION SUBTOTAL	DIVISION TOTAL
1	**GENERAL CONDITIONS**		**$ 36,793**
	Personnel	15,600	
	Small Tools	600	
	Temporary Facilities	2,438	
	Temporary Utilities	3,900	
	Clean - Up	7,655	
	Debris Removal	4,000	
	Testing & Inspections	2,600	
2	**SITEWORK**		**$ 9,500**
	Grading	9,500	
3	**CONCRETE**		**$ 12,275**
	Foundations	6,490	
	Slab on Grade	5,785	
6	**CARPENTRY**		**$ 47,168**
	Rough Carpentry	31,298	
	Finish Carpentry	2,671	
	Cabinetry	13,199	
7	**MOISTURE PROTECTION**		**$ 9,081**
	Insulation	3,617	
	Roofing	5,465	
8	**DOORS, WINDOWS, & GLASS**		**$ 9,930**
	Wood Doors and Frames	2,518	
	Sliding Glass Door	2,610	
	Garage Door	1,162	
	Finish Hardware	853	
	Windows	2,787	
9	**FINISHES**		**$ 38,760**
	Lath & Plaster	7,889	
	Drywall	9,060	
	Hardwood Flooring	882	
	Ceramic Tile	7,545	
	Carpeting	4,544	
	Painting	8,841	
10	**SPECIALTIES**		**$ 542**
	Toilet Accessories	542	
11	**EQUIPMENT**		**$ 1,470**
	Appliances	1,470	
15	**MECHANICAL**		**$ 17,682**
	Plumbing	13,487	
	HVAC	4,195	
16	**ELECTRICAL**		**$ 8,150**
	Electrical Devices	8,150	
	SUBTOTAL	191,349	**191,349**
	Permits		**1,435**
	Contractors overhead, profit, taxes, etc.		**28,918**
	TOTAL CONSTRUCTION		**221,702**

Table F-8:
Construction Cost Estimate, Townhouse, Limited-Drift Redesign

DIVISION	DESCRIPTION	DIVISION SUBTOTAL	DIVISION TOTAL
1	**GENERAL CONDITIONS**		**$ 71,123**
	Personnel	48,000	
	Small Tools	500	
	Temporary Facilities	3,625	
	Temporary Utilities	4,650	
	Clean – Up	5,348	
	Debris Removal	6,400	
	Testing & Inspections	2,600	
2	**SITEWORK**		**$ 13,925**
	Grading	13,925	
3	**CONCRETE**		**$ 20,180**
	Foundations	9,260	
	Slab on Grade	10,920	
6	**CARPENTRY**		**$ 99,397**
	Rough Carpentry	73,471	
	Finish Carpentry	5,510	
	Cabinetry	20,416	
7	**MOISTURE PROTECTION**		**$ 22,406**
	Insulation	10,814	
	Roofing	11,593	
8	**DOORS, WINDOWS, & GLASS**		**$ 26,918**
	Wood Doors and Frames	6,678	
	Sliding Glass Door	6,375	
	Garage Door	2,181	
	Finish Hardware	2,375	
	Windows	9,309	
9	**FINISHES**		**$ 92,645**
	Lath & Plaster	19,384	
	Drywall	24,713	
	Ceramic Tile	13,744	
	Carpeting	8,906	
	Painting	25,897	
10	**SPECIALTIES**		**$ 3,200**
	Toilet Accessories	3,200	
11	**EQUIPMENT**		**$ 4,410**
	Appliances	4,410	
15	**MECHANICAL**		**$ 54,052**
	Plumbing	39,247	
	HVAC	14,805	
16	**ELECTRICAL**		**$ 19,485**
	Electrical Devices	19,485	
	SUBTOTAL	427,741	**427,741**
	Contractors overhead, profit, taxes, etc.		**65,123**
	Building Permit		**6,416**
	TOTAL		**$ 499,280**

Table F-9:
Construction Cost Estimate, Apartment, Steel Moment Frames

DIVISION	DESCRIPTION	DIVISION SUBTOTAL	DIVISION TOTAL
1	**GENERAL CONDITIONS**		**$ 103,189**
	Personnel	66,000	
	Small Tools	1,500	
	Temporary Facilities	4,456	
	Temporary Utilities	6,650	
	Clean – Up	13,982	
	Debris Removal	8,000	
	Testing & Inspections	2,600	
2	**SITEWORK**		**$ 15,827**
	Grading	14,083	
	Retrofit Demolition	**1,744**	
3	**CONCRETE**		**$ 32,319**
	Foundations	10,160	
	Retrofit Footing and Slab	**7,248**	
	Slab on Grade	14,911	
4	**MASONRY**		**$ -**
	None		
5	**METALS**		**$ 28,380**
	Structural Steel	17,300	
	Retrofit Moment Frames	**11,080**	
6	**CARPENTRY**		**$ 167,450**
	Rough Carpentry	135,416	
	Retrofit Rough Carpentry	**1,850**	
	Finish Carpentry	7,659	
	Cabinetry	22,525	
7	**MOISTURE PROTECTION**		**$ 15,879**
	Insulation	8,994	
	Roofing	6,885	
8	**DOORS, WINDOWS, & GLASS**		**$ 48,609**
	Wood Doors and Frames	13,060	
	Finish Hardware	9,143	
	Windows	26,406	
9	**FINISHES**		**$ 160,119**
	Lath & Plaster	35,569	
	Retrofit Lath & Plaster	**2,926**	
	Drywall	44,139	
	Ceramic Tile	14,640	
	Carpeting	15,725	
	Vinyl Flooring	3,677	
	Painting	43,443	
10	**SPECIALTIES**		**$ 1,955**
	Toilet Accessories	1,955	
11	**EQUIPMENT**		**$ 9,600**
	Appliances	9,600	
15	**MECHANICAL**		**$ 83,652**
	Plumbing	72,652	
	HVAC	11,000	
16	**ELECTRICAL**		**$ 40,840**
	Electrical Devices	40,840	
	SUBTOTAL	707,818	**707,818**
	Building Permit (1.5%)		**10,617**
	Contractors overhead, profit, taxes, etc.		**107,765**
	TOTAL		**$ 826,201**

Table F-10:
Construction Cost Estimate, Apartment, Shearwalls

DIVISION	DESCRIPTION	DIVISION SUBTOTAL	DIVISION TOTAL
1	**GENERAL CONDITIONS**		**$ 103,189**
	Personnel	66,000	
	Small Tools	1,500	
	Temporary Facilities	4,456	
	Temporary Utilities	6,650	
	Clean - Up	13,982	
	Debris Removal	8,000	
	Testing & Inspections	2,600	
2	**SITEWORK**		**$ 14,878**
	Grading	14,083	
	Retrofit Demolition	**795**	
3	**CONCRETE**		**$ 25,071**
	Foundations	10,160	
	Sab on Grade	14,911	
4	**MASONRY**		**$ -**
	None		
5	**METALS**		**$ 17,300**
	Structural Steel	17,300	
6	**CARPENTRY**		**$ 172,571**
	Rough Carpentry	135,416	
	Retrofit Rough Carpentry	6,851	
	Finish Carpentry	7,659	
	Retrofit Door Casing	**120**	
	Cabinetry	22,525	
7	**MOISTURE PROTECTION**		**$ 15,879**
	Insulation	8,994	
	Roofing	6,885	
8	**DOORS, WINDOWS, & GLASS**		**$ 48,609**
	Wood Doors and Frames	13,060	
	Finish Hardware	9,143	
	Windows	26,406	
9	**FINISHES**		**$ 159,131**
	Lath & Plaster	35,569	
	Drywall	44,139	
	Retrofit Drywall	**1,288**	
	Ceramic Tile	14,640	
	Carpeting	15,725	
	Vinyl Flooring	3,677	
	Painting	43,443	
	Retrofit Painting	**650**	
10	**SPECIALTIES**		**$ 1,955**
	Toilet Accessories	1,955	
11	**EQUIPMENT**		**$ 9,600**
	Appliances	9,600	
15	**MECHANICAL**		**$ 83,652**
	Plumbing	72,652	
	HVAC	11,000	
16	**ELECTRICAL**		**$ 40,840**
	Electrical Devices	40,840	
	SUBTOTAL	692,674	**692,674**
	Contractors overhead, profit, taxes, etc.		**105,460**
	Building Permit (1.5%)		**10,390**
	TOTAL		**$ 808,524**

Table F-11:
Summary of Unit Costs for Assembly Repair

Assembly	Description	D	Repair	Unit	x_m	β	Sheet
4.5.110.2101.01	Exterior shearwall, 3/8 C-D ply, 2x4, 16" OC, 7/8" stucco ext, no int finish	1	Re-nail, patch stucco	64 sf	131	0.2	6BA
		2	Demolish and replace	64 sf	742	0.2	
4.5.110.2101.02	Exterior shearwall, 15/32 C-D ply, 2x4, 16" OC, 7/8" stucco ext, no int finish	1	Re-nail, patch stucco	64 sf	131	0.2	6BA
		2	Demolish and replace	64 sf	742	0.2	6CA
4.5.110.2111.01	Exterior shearwall, 7/16 OSB, 2x4, 16" OC, 7/8" stucco ext, no int finish	1	Re-nail, patch stucco	64 sf	131	0.2	6BA
		2	Demolish and replace	64 sf	742	0.2	6CA
4.5.110.2501.01	Exterior wall, no structural sheathing, 2x4, 16" OC, 7/8" stucco ext, no int finish	1	Patch stucco	64 sf	100	0.2	1AA
		2	Demolish and replace	64 sf	513	0.2	1CA
4.6.152.1700.01	Doors, sliding, patio, aluminum, standard, 6'-0"x6'-8", wood frame, insulated glass	1	Replace	ea	190	0.2	7D
4.7.100.3001.01	Windows, wood, dbl hung, std glass, 3'-1.5"x4'	1	Replace	pane	178	0.2	3
4.7.110.6600.01	Window, Al frame, sliding, std glass, <25 sf	1	Replace	pane	120	0.3	7B
4.7.110.6609.01	Window, Al frame, fixed, std glass, 80"x80"	1	Replace	pane	120	0.3	
6.1.510.1202.01	GWB partition, no structural sheathing, 1/2" GWB one side, 2x4, 16" OC	1	Patch	64 sf	88	0.2	2AA
		2	Patch	64 sf	88	0.2	2AA
		3	Demolish and replace	64 sf	352	0.3	2CA
6.1.510.1203.01	GWB finish, 1/2", one side, on 2x4, 16"OC	1	Patch	64 sf	88	0.2	1AB
		2	Patch	64 sf	88	0.2	1AB
		3	Demolish and replace	64 sf	184	0.3	1CB
6.1.520.1201.01	Interior shearwall, 3/8 C-D ply, 2x4, 16" OC, 1/2" GWB finish one side	1	Patch	64 sf	88	0.2	8AA
		2	Patch	64 sf	88	0.2	8AA
		3	Remove drywall, re-nail, replace drywall	64 sf	185	0.3	8CC
		4	Demolish and replace	64 sf	445	0.3	8CA
6.1.520.1201.02	Interior shearwall, 15/32 C-D ply, 2x4, 16" OC, 1/2" GWB finish one side	1	Patch	64 sf	88	0.2	8AA
		2	Patch	64 sf	88	0.2	8AA
		3	Remove drywall, re-nail, replace drywall	64 sf	185	0.3	8CC
		4	Demolish and replace	64 sf	455	0.3	8CE
6.1.520.1202.01	Interior sheathing, 3/8 C-D ply, 1/2" GWB finish one side, on 2x4 16" OC	1	Patch	64 sf	88	0.2	8AA
		2	Remove drywall, re-nail, replace drywall	64 sf	185	0.3	8C
		3	Demolish and replace	64 sf	326	0.3	8CD
6.1.520.1202.02	Interior sheathing, 15/32 C-D ply, 1/2" GWB finish one side, on 2x4, 16" OC	1	Patch	64 sf	88	0.2	8A
		2	Remove drywall, re-nail, replace drywall	64 sf	185	0.3	8C
		3	Demolish and replace	64 sf	336	0.3	8CF
6.1.520.1211.01	Interior shearwall, 7/16 OSB, 2x4, 16" OC, 1/2" GWB finish one side	1	Patch	sf	88	0.2	8AA
		2	Patch	sf	88	0.2	8AA
		3	Remove drywall, re-nail, replace drywall	sf	185	0.3	8CC
		4	Demolish and replace	sf	455	0.3	8CA
6.1.520.1212.01	Interior sheathing, 7/16 OSB, 1/2" GWB finish one side, on 2x4 16" OC	1	Patch	sf	88	0.2	
		2	Remove drywall, re-nail, replace drywall	sf	185	0.3	
		3	Demolish and replace	sf	336	0.3	
8.1.160.1820.01	Electric water heater, res., 100F rise, 50 gal, 9 kW, 37 gph	1	Replace	ea	650	0.2	5
09910.700.1400	Paint on exterior stucco or concrete	1	Paint	sf	0.81	0.2	1AC
09910.920.0840	Paint on interior concrete, drywall, or plaster	1	Paint	sf	0.81	0.2	1AC
	Restore collapsed small house	1	Various	ea	$39-51k		9

Cost Data Sheet 1A

Assembly Type

4.5.110.2500.01, Stucco wall, cement stucco, 7/8", no ext sheath, mtl lath, 1/2" gyp int, on stud wall, 2x4, 16"OC. Unit: 64 sf (8 lf of 8' high wall)

Damage State

Light: light cracking of stucco, requires patching and touch-up paint. Light damage to wallboard, requires tape, mud, sanding, touch-up paint.

Repairs Recommended

Remove loose debris on both sides of wall. Deepen cracks with a hand tool in order to receive filler material. Fill stucco cracks with a combination of bonder material and stucco and texture over the cracked area to blend in. The drywall cracks will require joint compound and re-texturing to blend in. Paint touch-up will be inadequate so completely painting the interior wall will be required. Matching the stucco color of this age will be impossible so painting the entire wall will be required.

Labor

Drywall: 2 hours @ $36-$42 per hour	Plaster: 2 hours @ $40-$45 per hour
Painting: 2 hours @ $36-$42 per hour	

Materials and Equipment

Drywall: $10	Plaster: $15
Painting: $25	

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $291	Upper bound: $308	Lower bound: $274
Duration: Mean: 7.5 hours	Upper bound: 9 hours	Lower bound: 6 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day.

Cost Summary (by Task 4.1)

Cost code 1AA: stucco only, no paint: $85 + $15 = $100
Cost code 1AB: drywall only, no paint:: $78 + $10 = $88
Cost code 1AC: paint only: $78 + 25 = $103/128 sf = $0.81/sf
Check total $100 + 88 + 103 = $291 OK

Cost Data Sheet 1B

Assembly Type

4.5.110.2500.01, Stucco wall, cement stucco, 7/8", no ext sheath, mtl lath, 1/2" gyp int, on stud wall, 2x4, 16"OC. Unit: 64 sf (8 lf of 8' high wall)

Damage State

Moderate: heavy cracking of stucco, requires replacement. Moderate damage to wallboard, requires patching, tape, mud, extensive painting (i.e., a paint crew).

Repairs Recommended

Remove loose debris on both sides of wall. Deepen cracks with a hand tool in order to receive filler material. Fill stucco cracks with a combination of bonder material and stucco and texture over the cracked area to blend in. The drywall cracks will require joint compound and re-texturing to blend in. Paint touch up will be inadequate so completely painting the interior wall will be required. Matching the stucco color of this age will be impossible so painting the entire wall will be required.

Labor

Drywall: 2 hours @ $36-$42 per hour	Plaster: 2 hours @ $40-$45 per hour
Painting: 2 hours @ $36-$42 per hour	

Materials and Equipment

Drywall: $10	Plaster: $15
Painting: $25	

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $291	Upper bound: $308	Lower bound: $274
Duration: Mean: 7.5 hours	Upper bound: 9 hours	Lower bound: 6 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day.

Cost Summary (by Task 4.1)

Same as 1A.

Cost Data Sheet 1C

Assembly Type

4.5.110.2500.01, Stucco wall, cement stucco, 7/8", no ext sheath, mtl lath, 1/2" gyp int, on stud wall, 2x4, 16"OC. Unit: 64 sf (8 lf of 8' high wall)

Damage State

Severe: Heavy damage to stucco and wallboard, damage to stud connections; requires demolition and replacement of wall.

Repairs Recommended

Remove drywall, stucco, insulation and wall framing. Replace with new 2x4 Doug. Fir studs and plates along with new ½" drywall, tape and texture, R-13 insulation and stucco. Paint both sides of the entire wall, 2 coats at the new drywall and 1 coat on the existing wall areas that tie in.

Labor

Demolition: \$2.20 per sf. 4 hours @ \$30-\$35	Framing: \$1.20 per sf. 2 hours @ \$40-\$45
Drywall: \$ 1.39 per sf. 3 hours @ \$36-\$42	Painting: \$1.13 per sf. 2.5 hours @ \$36-\$42
Plaster: \$3.00 per sf. 5 hours @ \$40-\$45	Insulation: \$.18 per sf .5 hours @ \$30-\$35

Materials and Equipment

Demolition: \$0	Framing: \$.67 per sf.
Drywall: \$.39 per sf.	Painting: \$.39 per sf.
Plaster: \$.75 per sf	Insulation: \$.31 per sf

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: \$818.89	Upper bound: \$864.14	Lower bound: \$773.64
Duration: Mean: 18.5 hours	Upper bound: 20 hours	Lower bound: 17 hours

Comments, References

Demolition includes removal of drywall, wood framing, electrical wire, protecting adjacent areas and clean up. Add \$.76/sf to haul debris to dump site within 6 miles of project. (Excludes dump fees). Add \$45 per electrical outlet or switch. All unit costs are for repair, not new construction.

Cost Summary (by Task 4.1)

Cost code 1CA: framing, stucco, and insulation only. Assume demolition of stucco and framing accounts for 75% of the total demolition; debris removal accounts for 75% of total.

$$\text{Cost} = (0.75*2.20+1.20+3.00+0.18+0.67+0.75+0.31+0.75*0.76)*64 \text{ sf} = \$513$$

Cost code 1CB: drywall only, no paint, and ½ an outlet or switch, assuming that demolition and debris removal of drywall accounts for the other 25% of demolition and debris removal:

$$\text{Cost} = (0.25*2.20 + 1.39 + 0.39 + 0.25*0.76)*64 + 22.50 = \$184$$

Cost code 1CC: paint: Cost = (1.13 + 0.39)*128 = $195

Check total: 513 + 184 + 195 = 892. Note that original figure of 819 excludes debris removal and ½ a switch, costing 0.76*64 + 22.50 = 71. 819 + 71 = 890. OK.

Cost Data Sheet 2A

Assembly Type

6.1.510.1201.01, Drywall partition w/o base layer, wood stud framing, 1/2" both sides, 2x4, 16" OC. Unit: 64 sf (8 lf of 8' high wall)

Damage State

Light: Light damage to wallboard, requires tape, mud, sanding, touch-up paint (both sides).

Repairs Recommended

Remove cracked or loose mud, tape and drywall. Install new tape, joint compound and texture affected area. It is unlikely that touch up paint will be adequate; so painting the entire wall will be required.

Labor

Drywall: 4 hours @ $36-$42 per hour Painting: 3 hours @ $36-$42 per hour

Materials and Equipment

Drywall: $20 Painting: $45

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $338	Upper bound: $359	Lower bound: $317
Duration: Mean: 8 hours	Upper bound: 9 hours	Lower bound: 7 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day.

Cost Summary (by Task 4.1)

Patching wallboard one side only, excluding paint: Cost code 2AA: 2 hr*$39/hr + $10 = $88
Paint: Cost code 2AB: 3hr*$39/hr + $45 = $162

Cost Data Sheet 2B

Assembly Type

6.1.510.1201.01, Drywall partition w/o base layer, wood stud framing, 1/2" both sides, 2x4, 16" OC. Unit: 64 sf (8 lf of 8' high wall).

Damage State

Moderate: Moderate damage to wallboard, requires patching, tape, mud, sand, and repaint wall (both sides).

Repairs Recommended

Remove cracked or loose mud, tape and drywall. Install new tape, joint compound and texture affected area. It is unlikely that touch up paint will be adequate; so painting the entire wall will be required.

Labor

Drywall: 4 hours @ $36-$42 per hour   Painting: 3 hours @ $36-$42 per hour

Materials and Equipment

Drywall: $20   Painting: $45

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $338	Upper bound: $359	Lower bound: $317
Duration: Mean: 8 hours	Upper bound: 9 hours	Lower bound: 7 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day.

Cost Data Sheet 2C

Assembly Type

6.1.510.1201.01, Drywall partition w/o base layer, wood stud framing, 1/2" both sides, 2x4, 16" OC. Unit: 64 sf (8 lf of 8' high wall)

Damage State

Severe: Heavy damage to wallboard and damage to stud connections; requires demolition and replacement of wall.

Repairs Recommended

Remove all drywall and wall framing and replace with new 2x4 Doug. Fir studs and plates along with new ½" drywall, tape and texture. Both sides of the entire wall should be painted, 2 coats at the new drywall and 1 coat on the existing wall areas that tie in.

Labor

Demolition: $1.74 per sf. 3.5 hours @ $30-$35 Framing: $1.20 per sf. 2 hours @ $40-$45
Drywall: $ 1.39 per sf. 6.5 hours @ $36-$42 Painting: $1.13 per sf. 3 hours @ $36-$42

Materials and Equipment

Demolition: $0 Framing: $.67 per sf.
Drywall: $.78 per sf Painting: $.39 per sf.

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $687.01 Upper bound: $729.26 Lower bound: $644.76
Duration: Mean: 16 hours Upper bound: 17 hours Lower bound: 15 hours

Comments, References

Demolition of wall assemblies includes removal of drywall, wood framing, electrical wire as necessary, protecting adjacent areas and normal clean up. Add $.76 per sq. ft. to haul off debris to dump site within 6 miles of project. (Excludes dump fees). Add $45 per each electrical outlet or switch. All sf unit costs should be used for repair costs not new construction.

Cost Summary (by Task 4.1)

Consider demolition, debris removal, framing, drywall 1 side, ½ outlet or switch, no paint:
Cost code 2CA: (1.74 + 0.76 + 1.20 + 0.67 + 0.78)*64 + 22.50 = $352
Consider paint only: (1.13 + 0.39)*128 = 195

Cost Data Sheet 3

Assembly Type

4.7.100.3001.01, Windows, Wood, double hung, standard glass, 3’-1.5”x4’. Unit: ea

Damage State

Cracked glass

Repairs Recommended

Remove glazing and glass. Replace with 3/16” standard glass and glazing. Paint glazing sash as required.

Labor

Glazier: 1 hour @ $36-$42 per hour

Painting: 1.5 hours @ $36-$42 per hour

Materials and Equipment

Glass and Glazing: $65-$75

Painting: $10

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $177.50	Upper bound: $190	Lower bound: $165
Duration: Mean: 3 hours	Upper bound: 3.5 hours	Lower bound: 2.5 hours

Comments, References

Use a minimum fee of $100 for glass and glazing. Use a minimum fee of $85 for painting.

Cost Data Sheet 4A

Assembly Type

6.7.100.5101.01, Drywall ceiling, 5/8" FR drywall, on 2"x6" rafters, 16" OC. Unit: 64 sf

Damage State

Light: Light damage to wallboard, requires tape, mud, sanding, touch-up paint.

Repairs Recommended

Remove cracked or loose mud, tape and drywall. Install new tape, joint compound and texture affected area. It is unlikely that touch up paint will be adequate; so painting the entire ceiling will be required.

Labor

Drywall: 2.5 hours @ $36-$42 per hour

Painting: 1.5 hours @ $36-$42 per hour

Materials and Equipment

Drywall: $10

Painting: $17

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $183	Upper bound: $195	Lower bound: $171
Duration: Mean: 5 hours	Upper bound: 6 hours	Lower bound: 4 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day.

Cost Data Sheet 4B

Assembly Type

6.7.100.5101.01, Drywall ceiling, 5/8" FR drywall, on 2"x6" rafters, 16" OC. Unit: 64 sf

Damage State

Moderate: Moderate damage to wallboard, requires patching, tape, mud, sand, and repaint ceiling.

Repairs Recommended

Remove cracked or loose mud, tape and drywall. Install new tape, joint compound and texture affected area. It is unlikely that touch up paint will be adequate; so painting the entire ceiling will be required.

Labor

Drywall: 2.5 hours @ $36-$42 per hour

Painting: 1.5 hours @ $36-$42 per hour

Materials and Equipment

Drywall: $10

Painting: $17

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $183	Upper bound: $195	Lower bound: $171
Duration: Mean: 5 hours	Upper bound: 6 hours	Lower bound: 4 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day.

Cost Data Sheet 4C

Assembly Type

6.7.100.5101.01, Drywall ceiling, 5/8" FR drywall, on 2"x6" rafters, 16" OC. Unit:64 sf

Damage State

Severe: Heavy damage to wallboard; requires demolition and replacement of ceiling. No damage to rafters.

Repairs Recommended

Remove damaged drywall. Replace 5/8" fire resistant drywall, install with screws and tape and texture; paint with 2 coats of rolled on acrylic latex.

Labor

Demolition: $.59 per sf. 1.25 hours @ $30-$35 Drywall: $ 1.39 per sf. 3.5 hours @ $36-$42
Painting: $1.13 per sf. 2 hours @ $36-$42

Materials and Equipment

Demolition: $0 Drywall: $.39 per sq. ft
Painting: $.39 per sq. ft.

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $305.04	Upper bound: $324.67	Lower bound: $285.42
Duration: Mean: 7.37 hours	Upper bound: 8 hours	Lower bound: 6.75 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day. Demolition of ceiling assemblies include: removal of drywall, protecting adjacent areas and normal clean up. Add $.66 per sq. ft. to haul off debris to dump site within 6 miles of project. (Excludes dump fees.) All sf unit costs should be used for repair costs not new construction. All costs exclude contractor's overhead and profit, which can be calculated by multiplying the total project cost by 15% to 20% as an average range.

Cost Data Sheet 5

Assembly Type

8.1.160.1820.01, Electric water heater, residential, 100F rise, 50 gal, 9 kW 37 GPH. Unit: ea

Damage State

Overturned

Repairs Recommended

Replace water heater with a new 50-gallon electric water heater. Replace 3 copper water fittings and re-attach vent. (Assuming vent is not damaged) Re-wire electrical.

Labor

Plumbing: 3 hours @ $60-$70

Electrical: 1.5 hours @ $60-$70

Materials and Equipment

Plumbing: $22+(water heater)$244 = $266

Electrical: $10

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $561	Upper bound: $591	Lower bound: $531
Duration: Mean: 5.25 hours	Upper bound: 6 hours	Lower bound: 4.5

Comments, References

A more likely minimum fee for a complete replacement would be $600. Add $50 minimum to dump old water heater. If the existing water heater is salvageable, deduct $266. The new water heater used in this illustration has a 9 yr warranty, add $58 to use a 12 yr warranty.

Cost Data Sheet 6A

Assembly Type

4.5.110.2100.13, Exterior wall, 3/8 C-D ply, 2x4, 16" OC, 8d@6" edge, 8d@12" int, 1/2" gyp int, insul, 7/8" stucco ext, 5/8" AB@32" OC. Unit: 64 sf (8 lf of 8' high wall)

Damage State

Light: light cracking of stucco, requires patching and touch-up paint. Light damage to wallboard, requires mud, sanding, touch-up paint.

Repairs Recommended

Remove loose debris on both sides of wall. Deepen cracks with a hand tool in order to receive filler material. Fill stucco cracks with a combination of bonder material and stucco and texture over the cracked area to blend in. The drywall cracks will require joint compound and re-texturing to blend in. Paint touch up will be inadequate so completely painting the interior wall will be required. Matching the stucco color of this age will be impossible so painting the entire wall will be required.

Labor

Drywall: 2 hours @ $36-$42 per hour
Plaster: 2.5 hours @ $40-$45 per hour
Painting: 2.5 hours @ $36-$42 per hour

Materials and Equipment

Drywall: $10
Plaster: $15
Painting: $25

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $331.75	Upper bound: $351.50	Lower bound: $312
Duration: Mean: 9 hours	Upper bound: 10.5 hours	Lower bound: 7.5 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day. One half hour has been added to each applicable trade for additional time incurred on a hillside application where access may slow down production.

Cost Data Sheet 6B

Assembly Type

4.5.110.2100.13, Exterior wall, 3/8 C-D ply, 2x4, 16" OC, 8d@6" edge, 8d@12" int, 1/2" gyp int, insul, 7/8" stucco ext, 5/8" AB@32" OC. Unit: 64 sf (8 lf of 8' high wall)

Damage State

Moderate: heavy cracking of stucco, requires replacement. Moderate damage to wallboard, requires patching, tape, mud, extensive painting (i.e., a paint crew).

Repairs Recommended

Remove loose debris on both sides of wall. Deepen cracks with a hand tool to receive filler material. Fill stucco cracks with a combination of bonder material and stucco and texture over the cracked area to blend in. Drywall cracks require joint compound and re-texturing to blend in. Paint touch up will be inadequate so completely painting the interior wall will be required. Matching the stucco color of this age will be impossible so paint the entire wall.

Labor

Drywall: 2 hours @ $36-$42 per hour
Plaster: 2.5 hours @ $40-$45 per hour
Painting: 2.5 hours @ $36-$42 per hour

Materials and Equipment

Drywall: $10
Plaster: $15
Painting: $25

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $331.75	Upper bound: $351.50	Lower bound: $312
Duration: Mean: 9 hours	Upper bound: 10.5 hours	Lower bound: 7.5 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day. One half hour has been added to each applicable trade for additional time incurred on a hillside application where access may slow down production.

Cost Summary (by Task 4.1)

Consider the damage state that the sheathing-to-framing nailing has begun to fracture. Repair entails removing the stucco at the fractured edge, re-nailing, and replacing the stucco. Exclude painting, and add $10 to re-nail the sheathing:

Cost code 6BA: Stucco labor + materials, no painting, plus 10 = 121 + 10 = 131

Cost Data Sheet 6C

Assembly Type

4.5.110.2100.13, Exterior wall, 3/8 C-D ply, 2x4, 16" OC, 8d@6" edge, 8d@12" int, 1/2" gyp int, insul, 7/8" stucco ext, 5/8" AB@32" OC. Unit: 64 sf (8 lf of 8’ high wall)

Damage State

Severe: Heavy damage to stucco and wallboard, nails tear though plywood sheathing; requires demolition and replacement of wall.

Repairs Recommended

Remove all drywall, stucco, insulation, plywood and wall framing and replace with new 2x4 Doug. Fir studs and plates, new 3/8” cdx plywood along with new ½” drywall, tape and texture, R-13 insulation and stucco. Both sides of the entire wall should be painted, 2 coats at the new drywall and 1 coat on the existing wall areas that tie in.

Labor

Demolition: $2.75 per sf. 5.5 hours @ $30-$35	Framing: $2.29 per sf. 3.5 hours @ $40-$45
Drywall: $ 1.39 per sf. 3 hours @ $36-$42	Painting: $1.13 per sf. 3 hours @ $36-$42
Plaster: $3.60 per sf. 5.5 hours @ $40-$45	Insulation: $.18 per sf .5 hours @ $30-$35

Materials and Equipment

Demolition: $0	Framing: $.96 per sf.
Drywall: $.39 per sf.	Painting: $.39 per sf.
Plaster: $.75 per sf	Insulation: $.31 per sf

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $990.45	Upper bound: $1,045.70	Lower bound: $935.20
Duration: Mean: 22.5 hours	Upper bound: 24 hours	Lower bound: 21 hours

Comments, References

Demolition of wall assemblies include: removal of drywall, plywood, wood framing, insulation, electrical wire as necessary, protecting adjacent areas and normal clean up. Add $.76 per sq. ft. to haul off debris to dump site within 6 miles of project. (Excludes dump fees.) One half hour has been added to each applicable trade for additional time incurred on a hillside application where access may slow down production. Add $45 per each electrical outlet or switch. All sf unit costs should be used for repair costs not new construction.

Cost Summary (by Task 4.1)

Consider only demolition, stucco, sheathing, framing, insulation, and excluding the drywall and painting:

Cost code 6CA: (2.75+2.29+3.60+0.18+0.96+0.75+0.31+0.76)*64 + 0.5*45 = $742

Consider the drywall only and ½ outlet or switch:

Cost code 6CB: (1.39+0.39)*64 + 0.5*45 = $136

Paint:

Cost code 6CC: (1.13+0.39)*128 = $195

Total = 742 + 136 + 195 = 1073 OK. (This includes debris removal, 0.76*64 and ½ a switch, 22.50, which the original 990.45 excluded. 990 + 0.76*64 + 22.50 = 1061 close enough.)

Cost Data Sheet 7A

Assembly Type

4.7.110.6600.01, Window, Al frame, sliding, standard glass, 3'x2'. Unit: ea

Damage State

Cracked glass (one pane)

Repairs Recommended

Remove glass. Replace with 1/8” standard glass.

Labor

Glazier: 1 hour @ $36-$42 per hour

Materials and Equipment

Glass: $35-$45

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $100	Upper bound: $129	Lower bound: $71
Duration: Mean: 1.5 hours	Upper bound: 2 hours	Lower bound: 1 hour

Comments, References

Use a minimum fee of $100 for glass.

Cost Data Sheet 7B

Assembly Type

4.7.110.6650.01, Window, Al frame, sliding, standard glass, 5'x3'. Unit: ea

Damage State

Cracked glass (one pane)

Repairs Recommended

Remove glazing and glass. Replace with 1/8” standard glass.

Labor

Glazier: 1.5 hours @ $36-$42 per hour

Materials and Equipment

Glass: $35-$45

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $119.50	Upper bound: $150	Lower bound: $89
Duration: Mean: 2 hours	Upper bound: 2.5 hours	Lower bound: 1.5 hours

Comments, References

Use a minimum fee of $100 for glass.

Cost Data Sheet 7C

Assembly Type

4.7.110.6700.01, Window, Al frame, sliding, standard glass, 8'x4'. Unit: ea.

Damage State

Cracked glass (one pane)

Repairs Recommended

Remove glazing. Replace with 1/8” standard glass.

Labor

Glazier: 2 hours @ $36-$42 per hour

Materials and Equipment

Glass: $45-$55

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $149	Upper bound: $181	Lower bound: $117
Duration: Mean: 2.5 hours	Upper bound: 3 hours	Lower bound: 2 hours

Comments, References

Use a minimum fee of $100 for glass.

Cost Data Sheet 7D

Assembly Type

4.6.152.1700.01, Doors, sliding, patio, aluminum, standard, 6'-0"x6'-8", with aluminum frame, tempered glass. Unit: ea.

Damage State

Cracked glass (one pane)

Repairs Recommended

Remove and replace with 1/8" standard tempered glass.

Labor

Glazier: 2 hours @ $36-$42 per hour

Materials and Equipment

Glass: $115

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $193	Upper bound: $199	Lower bound: $187
Duration: Mean: 2.5 hours	Upper bound: 3 hours	Lower bound: 2 hours

Comments, References

Use a minimum fee of $150 for glass. Add 10% for dual glazing.

Cost Data Sheet 8A

Assembly Type

6.1.520.1200.13, Interior wall, 3/8 C-D ply, 2x4, 16" OC, 8d@6" edge, 8d@12" int, 1/2" gyp finish ea side, 5/8" AB@32" OC. Unit: 64 sf (8 lf of 8' high wall)

Damage State

Light: light cracking of wallboard, requires mud, sanding, touch-up paint.

Repairs Recommended

Remove cracked or loose mud, tape and drywall. Install new tape, joint compound and texture affected area. It is unlikely that touch up paint will be adequate; so painting the entire wall will be required.

Labor

Drywall: 4 hours @ $36-$42 per hour

Painting: 3 hours @ $36-$42 per hour

Materials and Equipment

Drywall: $20

Painting: $45

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $338	Upper bound: $359	Lower bound: $317
Duration: Mean: 8 hours	Upper bound: 9 hours	Lower bound: 7 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day.

Cost Summary (by Task 4.1)

Separate drywall and painting, and consider one side of the drywall only:

8AA: Drywall only: 4*39 + 20 = 176 for 2 sides. For 1 only: 88.
8AB: Paint: 3*39+45 = 162 for 2 sides. For 1 only: 81
Check total: 176 + 162 = 338 OK

Cost Data Sheet 8B

Assembly Type

6.1.520.1200.13, Interior wall, 3/8 C-D ply, 2x4, 16" OC, 8d@6" edge, 8d@12" int, 1/2" gyp finish ea side, 5/8" AB@32" OC. Unit: 64 sf (8 lf of 8' high wall)

Damage State

Moderate damage to wallboard, requires patching, tape, mud, extensive painting (i.e., a paint crew).

Repairs Recommended

Remove cracked or loose mud, tape and drywall. Install new tape, joint compound and texture affected area. It is unlikely that touch up paint will be adequate; so painting the entire wall will be required.

Labor

Drywall: 4 hours @ $36-$42 per hour

Painting: 3 hours @ $36-$42 per hour

Materials and Equipment

Drywall: $20

Painting: $45

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $338	Upper bound: $359	Lower bound: $317
Duration: Mean: 8 hours	Upper bound: 9 hours	Lower bound: 7 hours

Comments, References

A handyman would perform a repair job of this size; there would typically be a minimum fee of $300 per day.

Cost Data Sheet 8C

Assembly Type

6.1.520.1200.13, Interior wall, 3/8 C-D ply, 2x4, 16" OC, 8d@6" edge, 8d@12" int, 1/2" gyp finish ea side, 5/8" AB@32" OC. Unit: 64 sf (8 lf of 8' high wall)

Damage State

Severe: Heavy damage to wallboard, nails tear though plywood sheathing; requires demolition and replacement of wall.

Repairs Recommended

Remove all drywall, plywood and wall framing and replace with new 2x4 Doug. Fir studs and plates, new 3/8" cdx plywood along with new ½" drywall, tape and texture. Both sides of the entire wall should be painted, 2 coats at the new drywall and 1 coat on the existing wall areas that tie in.

Labor

Demolition: $2.10/sf. 4.5 hours @ $30-$35
Drywall: $ 1.39/sf. 6.5 hours @ $36-$42
Framing: $2.29 per sf. 3.5 hours @ $40-$45
Painting: $1.13 per sf. 3 hours @ $36-$42

Materials and Equipment

Demolition: $0
Drywall: $.78 per sf
Framing: $.96 per sf.
Painting: $.39 per sf.

Total Repair Cost and Duration (Mean, 10th percentile, 90th percentile)

Cost: Mean: $826.78	Upper bound: $875.28	Lower bound: $778.28
Duration: Mean: 18.5 hours	Upper bound: 19.5 hours	Lower bound: 17.5 hours

Comments, References

Demolition of wall assemblies include: removal of drywall, plywood, wood framing, electrical wire as necessary, protecting adjacent areas and normal clean up. Add $.76 per sq. ft. to haul off debris to dump site within 6 miles of project. (Excludes dump fees.) Add $45 per each electrical outlet or switch. All sf unit costs should be used for repair costs not new construction.

Cost Summary (by Task 4.1)

Demolition, framing (which presumably includes plywood sheathing), drywall 1 side only, remove debris, ½ an outlet or switch, excluding painting. Assume that the demolition and debris removal of the framing, sheathing, and only 1 side of the drywall represents 75% of the total demolition and debris removal:

8CA: (0.75*$2.10/sf + 2.29/sf + 1.39/sf + 0.78/sf + 0.75*0.76/sf)*64 sf + 22.50 = $445

Painting: 8CB: (1.13 + 0.39)*128 = $195

Demolition of drywall on the other side, re-nailing the sheathing on that side, and replacement of that drywall:

8CC: (0.25*$2.10/sf + 1.39/sf + 0.78/sf + 0.25*0.76/sf)*64 sf = $185

Check: 445 + 195 + $187 = $827 OK

Demolition and replacement of sheathing and drywall on one side only, excluding demolition and replacement of the framing. Also exclude painting. Include ½ a switch or outlet. Assume that the demolition and debris removal of the sheathing and 1 side of the drywall represents 50% of the total demolition and debris removal, and that the labor and materials involved in demolition and installation the new sheathing represents 50% of the labor and material cost of the demolition and framing.

8CD: (0.5*$2.10/sf + 0.5*2.29/sf + 1.39/sf + 0.78/sf + 0.5*0.76/sf)*64 sf + 22.50 = $326

Same tasks but using 15/32 plywood, assuming material costs $0.13/sf more and labor costs $0.02/sf more, per RS Means Co., Inc., 2000, Repair and Remodeling Cost Data, Line Nos. 06160.800.0052 and 06160.800.0102:

8CE (similar to 8CA, but 15/32 ply): $445 + $0.15/sf*64 sf = $455
8CF (similar to 8CD, but 15/32 ply): $326 + 0.15/sf*64 sf = $336

Cost Data Sheet 9

This cost data sheet is very approximate, and is based on a rough picture of the potential damage resulting from cripple-wall collapse. Further study of this cost item is warranted.

Assembly Type

Collapsed small house.

Damage State

Cripple walls collapse, leaving first floor resting on the foundation. Electrical and plumbing hookups are fractured. Building above first floor is largely intact, with various damages to all the finishes.

Repairs Recommended

Raise the structure, rebuild the cripple wall with bracing, repair exterior stucco, patch interior drywall damage, repair broken ceramic tile, replace broken windows, reinstall carpet, and repaint.

Total Repair Cost and Duration

Raise structure:	$12,000
Rebuild and brace the cripple wall:	6,300
Stucco repairs:	3,500
Drywall repairs:	4,500
Ceramic tile:	2,300
Hookups, carpet, paint, glass, etc.:	9,400

Total cost, mean: $38,000 Upper bound: $43,000 Lower bound: $33,000

Comments, References

This cost is modeled as uniformly distributed between $33,000 and $43,000. Adding median contractor overhead and profit of 17.5% (being careful not to double-count O&P): $39,000 to $51,000.

Appendix G. Vulnerability Functions in Tabular Form

Warning. These tables are presented for the convenience of readers who have already reviewed the project objectives (Chapter 1), the methodology used to create these tables (Chapter 3), the particular, detailed buildings reflected here (Chapter 4), the sources and degree of uncertainty (Chapter 5), and the validation performed (Chapters 4 and 5). Other readers are advised not to use these tables.

The tables of mean damage factor as a function of S_a are sample means based on 20 simulations per variant per level of S_a. The tables of mean damage factor versus PGA are sample means based on a varying number of simulations per PGA level. Both sets of tables result from the same analyses.

Table G-1:
Small-House Mean Damage Factor as a Function of S_a[1].

S_a, g	Poor	Typical	Superior	Braced
0.1	0.000	0.000	0.000	0.000
0.2	0.005	0.000	0.000	0.000
0.3	0.033	0.004	0.000	0.000
0.4	0.118	0.012	0.000	0.005
0.5	0.177	0.061	0.000	0.009
0.6	0.222	0.126	0.000	0.048
0.7	0.246	0.166	0.000	0.054
0.8	0.240	0.190	0.002	0.103
0.9	0.239	0.186	0.002	0.098
1.0	0.211	0.163	0.004	0.115
1.1	0.238	0.191	0.007	0.147
1.2	0.261	0.230	0.009	0.174
1.3	0.251	0.198	0.012	0.149
1.4	0.251	0.215	0.021	0.174
1.5	0.258	0.204	0.024	0.193
1.6	0.284	0.193	0.017	0.127
1.7	0.258	0.237	0.024	0.219
1.8	0.231	0.204	0.026	0.136
1.9	0.241	0.181	0.023	0.126
2.0	0.311	0.247	0.036	0.171

Table G-2:
Small-House Mean Damage Factor as a Function of PGA[1].

PGA, g	Poor	Typical	Superior	Braced
0.1	0.001	0.000	0.000	0.000
0.2	0.022	0.003	0.000	0.001
0.3	0.065	0.009	0.000	0.006
0.4	0.151	0.035	0.000	0.011
0.5	0.205	0.097	0.001	0.029
0.6	0.229	0.148	0.002	0.049
0.7	0.249	0.183	0.005	0.093
0.8	0.262	0.194	0.009	0.120
0.9	0.266	0.237	0.018	0.175
1.0	0.269	0.246	0.018	0.192
1.1	0.302	0.279	0.027	0.249
1.2	0.331	0.313	0.029	0.247
1.3	0.337	0.303	0.040	0.282
1.4	0.333	0.341	0.058	0.280
1.5	0.344	0.249	0.050	0.260
1.6	0.365	0.320	0.029	0.366
1.7	0.304	0.295	0.033	0.321

(1) See warning on page G-1.

Table G-3:
Large-House Mean Damage Factor as a Function of S_a[(1)].

S_a, g	Poor	Typical	Superior	Waist wall	Rigid diaph.	IO
0.1	0.000	0.000	0.000	0.000	0.000	0.000
0.2	0.000	0.000	0.000	0.000	0.000	0.000
0.3	0.000	0.000	0.000	0.000	0.000	0.000
0.4	0.005	0.000	0.000	0.001	0.001	0.000
0.5	0.015	0.002	0.001	0.003	0.004	0.000
0.6	0.033	0.006	0.002	0.007	0.010	0.001
0.7	0.045	0.016	0.006	0.015	0.016	0.002
0.8	0.052	0.025	0.014	0.022	0.025	0.004
0.9	0.064	0.031	0.016	0.031	0.034	0.006
1.0	0.083	0.049	0.022	0.040	0.044	0.015
1.1	0.081	0.049	0.043	0.049	0.060	0.020
1.2	0.095	0.067	0.048	0.061	0.065	0.023
1.3	0.108	0.076	0.050	0.078	0.075	0.038
1.4	0.096	0.070	0.049	0.071	0.068	0.034
1.5	0.100	0.070	0.053	0.066	0.068	0.035
1.6	0.107	0.076	0.063	0.078	0.086	0.046
1.7	0.097	0.068	0.051	0.077	0.076	0.039
1.8	0.102	0.083	0.065	0.078	0.081	0.049
1.9	0.127	0.085	0.076	0.097	0.107	0.052
2.0	0.115	0.093	0.082	0.092	0.094	0.058

Table G-4:
Large-House Mean Damage Factor as a Function of PGA[(1)].

PGA, g	Poor	Typical	Superior	Waist wall	Rigid diaph.	IO
0.1	0.000	0.000	0.000	0.000	0.000	0.000
0.2	0.000	0.000	0.000	0.000	0.000	0.000
0.3	0.001	0.000	0.000	0.000	0.000	0.000
0.4	0.008	0.002	0.000	0.002	0.001	0.000
0.5	0.022	0.005	0.001	0.006	0.008	0.001
0.6	0.039	0.018	0.010	0.016	0.020	0.004
0.7	0.060	0.030	0.021	0.030	0.034	0.011
0.8	0.070	0.042	0.028	0.040	0.041	0.022
0.9	0.093	0.063	0.046	0.060	0.062	0.028
1.0	0.104	0.068	0.045	0.064	0.070	0.034
1.1	0.119	0.089	0.063	0.073	0.094	0.038
1.2	0.133	0.088	0.069	0.101	0.096	0.047
1.3	0.150	0.123	0.094	0.129	0.115	0.060
1.4	0.148	0.114	0.102	0.126	0.141	0.066

(1) See warning on page G-1.

Table G-5:
Townhouse Mean Damage Factor as a Function of S_a(1).

Sa, g	Poor	Typical	Superior	Limited drift
0.1	0.000	0.000	0.000	0.000
0.2	0.000	0.000	0.000	0.000
0.3	0.000	0.000	0.000	0.000
0.4	0.005	0.001	0.001	0.000
0.5	0.017	0.004	0.002	0.000
0.6	0.031	0.012	0.008	0.003
0.7	0.040	0.016	0.010	0.007
0.8	0.047	0.026	0.019	0.009
0.9	0.058	0.036	0.023	0.011
1.0	0.065	0.043	0.033	0.017
1.1	0.072	0.047	0.041	0.024
1.2	0.079	0.057	0.051	0.033
1.3	0.094	0.072	0.057	0.035
1.4	0.082	0.067	0.057	0.044
1.5	0.094	0.064	0.057	0.047
1.6	0.085	0.067	0.062	0.048
1.7	0.105	0.080	0.070	0.060
1.8	0.103	0.080	0.069	0.066
1.9	0.094	0.078	0.070	0.065
2.0	0.108	0.088	0.080	0.072

Table G-6:
Townhouse Mean Damage Factor as a Function of PGA(1).

PGA, g	Poor	Typical	Superior	Limited drift
0.1	0.000	0.000	0.000	0.000
0.2	0.000	0.000	0.000	0.000
0.3	0.003	0.000	0.000	0.000
0.4	0.007	0.002	0.002	0.001
0.5	0.024	0.011	0.007	0.004
0.6	0.034	0.020	0.013	0.011
0.7	0.052	0.033	0.023	0.015
0.8	0.061	0.041	0.035	0.031
0.9	0.080	0.051	0.045	0.032
1.0	0.088	0.062	0.056	0.042
1.1	0.105	0.079	0.069	0.048
1.2	0.117	0.091	0.081	0.055
1.3	0.128	0.108	0.080	0.069
1.4	0.128	0.108	0.089	0.082
1.5	0.143	0.110	0.109	0.080

(1) See warning on page G-1.

Table G-7: Apartment Mean Damage Factor as a Function of S_a[1].

Sa, g	Poor	Typical	Superior	Shearwall	Steel frames
0.1	0.1	0.000	0.000	0.000	0.000
0.2	0.2	0.004	0.004	0.001	0.002
0.3	0.3	0.011	0.008	0.006	0.006
0.4	0.4	0.016	0.014	0.011	0.009
0.5	0.5	0.026	0.023	0.013	0.014
0.6	0.6	0.083	0.030	0.023	0.020
0.7	0.7	0.132	0.081	0.074	0.024
0.8	0.8	0.132	0.131	0.031	0.026
0.9	0.9	0.188	0.042	0.032	0.034
1.0	1.0	0.237	0.137	0.082	0.037
1.1	1.1	0.380	0.237	0.137	0.051
1.2	1.2	0.291	0.334	0.190	0.149
1.3	1.3	0.293	0.290	0.235	0.103
1.4	1.4	0.149	0.195	0.140	0.100
1.5	1.5	0.531	0.437	0.289	0.119
1.6	1.6	0.533	0.579	0.296	0.211
1.7	1.7	0.291	0.195	0.144	0.062
1.8	1.8	0.389	0.389	0.336	0.204
1.9	1.9	0.436	0.571	0.432	0.160
2.0	2.0	0.250	0.287	0.238	0.111

Table G-8: Apartment Mean Damage Factor as a Function of PGA[1].

PHA, g	Poor	Typical	Superior	Shearwall	Steel frames
0.1	0.002	0.002	0.001	0.001	0.001
0.2	0.010	0.008	0.006	0.006	0.007
0.3	0.016	0.014	0.010	0.010	0.013
0.4	0.025	0.022	0.017	0.016	0.020
0.5	0.097	0.037	0.029	0.027	0.036
0.6	0.146	0.045	0.059	0.037	0.042
0.7	0.193	0.163	0.036	0.039	0.043
0.8	0.202	0.254	0.139	0.053	0.055
0.9	0.402	0.403	0.271	0.110	0.067
1.0	0.532	0.453	0.292	0.154	0.070
1.1	0.612	0.532	0.454	0.157	0.079
1.2	0.453	0.560	0.391	0.185	0.075
1.3	0.661	0.598	0.526	0.404	0.078
1.4	0.592	0.581	0.371	0.281	0.082

(1) See warning on page G-1.